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NATIONAL DAM SAFETY PROGRAM DASHVILLE DAM, INVENTORY NUMBER (NY--ETC(U)
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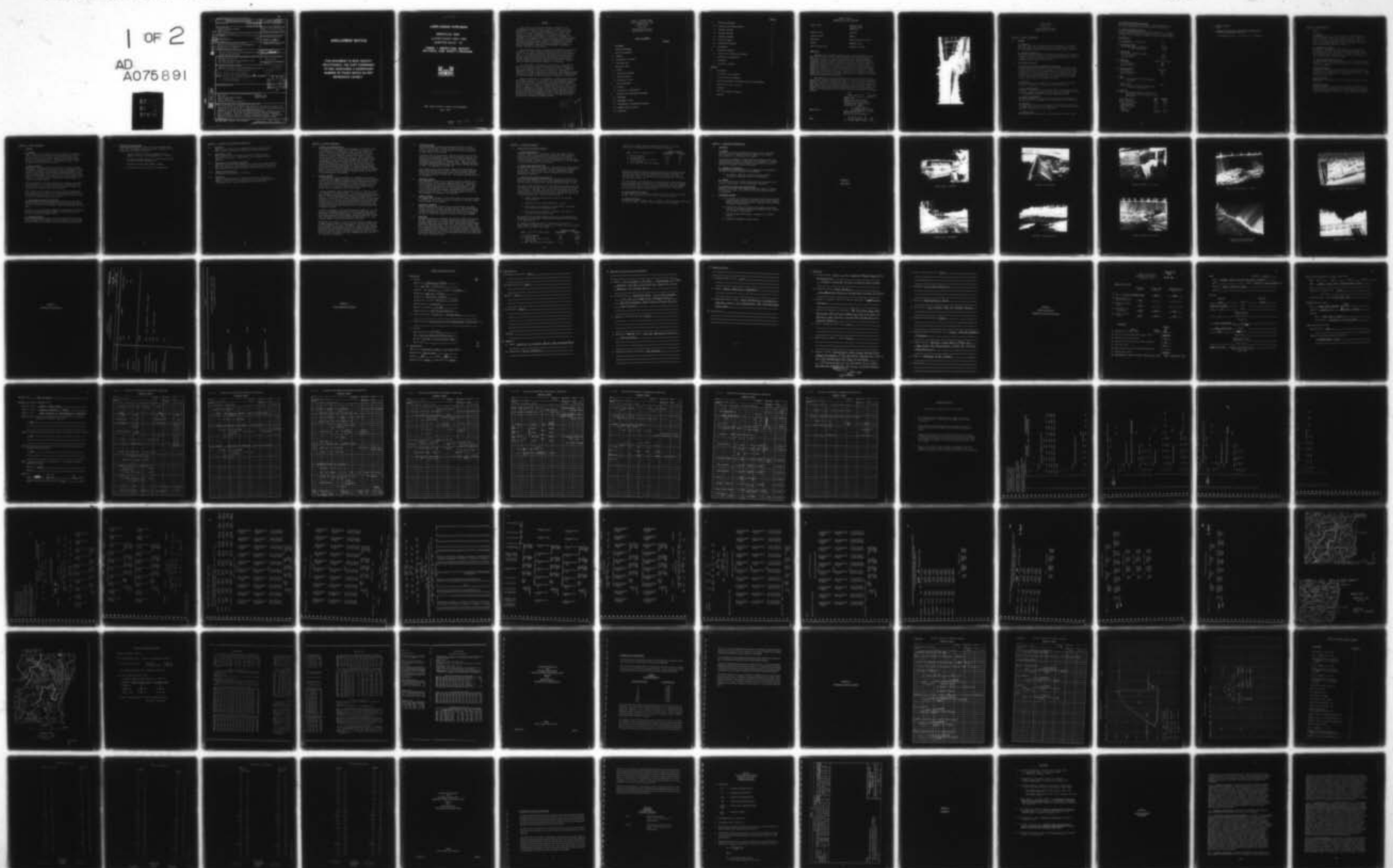

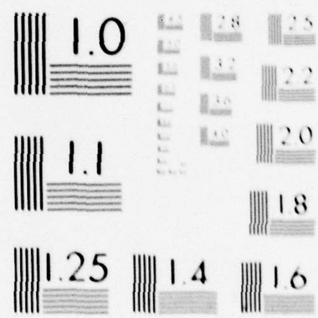

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REPORT DOCUMENTATION PAGE

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Dashville Dam
Ulster County

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report provides information and analysis on the physical condition of the dam as of the report date. Information and analysis are based on visual inspection of the dam by the performing organization.
Dashville Dam did not reveal any conditions which pose an immediate threat to life or property. Additional structural stability analysis recommended. Minor deficiencies limited to concrete surface deterioration. Insufficient spillway discharge capacity for passing ½ of PMF has deemed spillway inadequate.

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DISCLAIMER NOTICE

LOWER HUDSON RIVER BASIN

DASHVILLE DAM

ULSTER COUNTY, NEW YORK

INVENTORY NO. N.Y. 76

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

NEW YORK DISTRICT CORPS OF ENGINEERS

MAY, 1979

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through frequent inspections can unsafe conditions be detected and only through continued care and maintenance can these conditions be prevented or corrected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
DASHVILLE DAM
I.D. No. NY-76
(#759 L.H.)
LOWER HUDSON RIVER BASIN
ULSTER COUNTY, NEW YORK

TABLE OF CONTENTS

PHASE 1 REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam:	Dashville Dam I.D. No. NY-76 (#759-LH)
State Located:	New York
County Located:	Ulster
Watershed:	Lower Hudson River Basin
Stream:	Wallkill River
Date of Inspection:	November 13, 1978

ASSESSMENT

Examination of available documents and a visual inspection of the dam did not reveal conditions which constitute an immediate hazard to human life or property. However, additional studies are recommended to further evaluate conditions affecting the dam. Additional detailed structural stability analyses should be commenced within six months and completed within one year of the date of this report. Such analyses should be performed in accordance with the Corps of Engineers Guidelines, included in Appendix G. Appropriate remedial measures deemed necessary should be completed within two years of the date of this report. Minor deficiencies found during the visual inspection were limited to concrete surface deterioration and cracking. Such deficiencies should be corrected during normal maintenance operations.

The spillway, not having sufficient discharge capacity for passing one-half the Probable Maximum Flood (PMF), is considered to be inadequate. Because of relatively insignificant reservoir storage capacity at the dam, both dam non-failure and failure discharges routed over the spillway attain similar water surface elevations in the downstream areas. Hence, dam failure from overtopping would not significantly increase the hazard to loss of life downstream from that which would exist just before overtopping failure.

George Koch

George Koch
Chief, Dam Safety Section
New York State Department of
Environmental Conservation
NY License No. 45937

Approved By:

Col. Clark H. Benn
New York District Engineer

Date: 24 September 79

OVERVIEW - DASHVILLE DAM

DASHVILLE DAM
I.D. No. NY-76
(#759 L.H.)
LOWER HUDSON RIVER BASIN
ULSTER COUNTY, NEW YORK

SECTION 1: PROJECT INFORMATION

1.1 GENERAL

a. Authority

The Phase 1 inspection reported herein was authorized by the Department of the Army, New York District, Corps of Engineers, to fulfill the requirements of the National Dam Inspection Act, Public Law 92-367.

b. Purpose of Inspection

This inspection was conducted to evaluate the existing conditions of the dam, to identify deficiencies and hazardous conditions, to determine if these deficiencies constitute hazards to life and property, and to recommend remedial measures where required.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam and Appurtenant Structures

The Dashville Dam is a run-of-river, concrete gravity structure part of which forms an ogee section. The dam is 370 feet long and varies in height from 3 feet at the northwestern end to 38 feet at the southeastern end. There are flashboards, 3.5 feet in height, across the spillway crest. An operating hydroelectric power station is located on the southeastern end of the dam. Four gates in the power house control the flow into the hydromachinery units.

b. Location

The dam is located on the Wallkill River, approximately one quarter mile west of the hamlet of Dashville along State Route 213.

c. Size Classification

The dam is 38 feet high and the reservoir has a storage capacity of 92 acre-feet. Therefore, the dam is in the small size category as defined by the Recommended Guidelines for Safety Inspection of Dams.

d. Hazard Classification

The dam is classified as "significant" hazard due to the presence of several houses and the Sturgeon Pool Dam downstream of this structure.

e. Ownership

This dam is owned by the Central Hudson Gas and Electric Corporation of Poughkeepsie, New York. Mr. Donald Otis (914) 452-2000 is the representative of the utility who was contacted.

f. Purpose of Dam

The dam provides a storage pool for the hydroelectric power station.

g. Design and Construction History

The dam was designed in 1919 by the J.G. White Engineering Corporation of New York and constructed between 1920 and 1922.

h. Normal Operating Procedures

Water is discharged primarily through the power house. According to data provided by Central Hudson Gas and Electric, the average discharge is 60 cfs during the summer and 203 cfs during the winter.

1.3 PERTINENT DATA

a. Drainage Area (square miles) 765

b. Discharge at Dam	(cfs)
Top of Dam (without flashboards)	14,685
" " (with flashboards)	2,243

c. Elevations	
Top of Dam - Northwest Abutment	174.0
Top of Flashboards	172.5
Spillway Crest	169.0

d. Reservoir	Surface Area (acres)
Top of Dam	300
Top of Flashboards	300
Spillway Crest	9.6

e. Storage Capacity	(acre-feet)
Top of Dam	965
Top of Flashboards	515
Spillway Crest	92

f. Dam

Type: Concrete gravity with appurtenant structures

Length (feet) 370

Height Varies from 3 feet (Northwest End) to 38 feet (Southeast End)

g. Spillway

Type: Uncontrolled, gravity structure with ogee and non-ogee sections and 3.5 foot flashboards.

	Ogee	Non-Ogee
Weir Length (feet):	140	215
Crest Elevation:	169.0	169.0
Width @ Crest (feet):	—	1.5
Upstream radius	3.0	—
Downstream radius	33.8	—
Slopes (V : H)		
Upstream	Vertical	1:1.25
Downstream	1:0.64	1:1

h. Reservoir Drain

None

i. Appurtenant Structures - Hydroelectric Power Station

4 bays - each opening 10 feet wide

2 hydromachinery units - Total capacity of 2 megawatts

SECTION 2: ENGINEERING DATA

2.1 DESIGN

a. Geology

The Dashville Dam is located in the Wallkill Valley segment of the Hudson-Mohawk lowlands physiographic province of New York State. The valley is broad and covered with glacial drift. The bedrock in the area was formed during the Ordovician era and consists of interbedded greywacke, siltstone and shale. The rock strata has undergone significant folding.

b. Subsurface Information

No records of any subsurface investigations in the vicinity of this dam were available. The only information available was the data submitted with the application to construct the dam. This data indicates that the foundation for the dam is bedrock which is free of objectionable faults and seams.

c. Dam and Appurtenant Structures

The dam and power house were designed by the J.G. White Engineering Corporation. Copies of several drawings from the project have been included in Appendix H.

2.2 CONSTRUCTION RECORDS

Construction plans and some correspondence from 1919 are the only construction records available. The records indicate that the dam was constructed in 1920-1922 under the supervision of the J.G. White Engineering Corporation.

2.3 OPERATING RECORDS

There were no operating or water level records available for this structure.

2.4 EVALUATION OF DATA

The data presented in this report was obtained from the files of the Department of Environmental Conservation and from the Central Hudson Gas and Electric Corporation. The information available appears to be adequate and reliable for Phase 1 inspection purposes.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General

Visual inspection of the Dashville Dam was conducted on November 13, 1978. The weather was overcast with the temperature near 35° F. The water surface at the time of inspection was several inches above the crest of the spillway. Approximately 75 feet of flashboards on the northwestern end of the structure had been opened allowing flow over this part of the dam.

b. Dam - Spillway

Inspection of the main portion of the dam did not reveal any major deficiencies. There were several small areas of spalling, surface cracking and separation of the gunite near the downstream toe. Small sections of the gunite had been removed, exposing portions of the steel reinforcing mesh. Near the midpoint of the dam, water flowing along the bedrock foundation at the toe had scoured a channel in the concrete. A triangular section approximately 3 inches high and 3 inches deep had been removed.

With the exception of the minor spalling and cracking of the gunite, the downstream face of the dam appeared to be in satisfactory condition. However, since the face had been treated with gunite, the condition of the concrete could not be observed.

Water flowing over the spillway in the area where the flashboards were opened had scoured some of the backfill from beneath the northwest abutment forming a small void. The abutment is founded on rock, so this hole does not appear to be serious.

c. Appurtenant Structures - Powerhouse

Concrete surfaces on the powerhouse were spalling and deteriorated. The intake structure has been repaired several times in the past 50 years. At the time of the inspection, divers were repairing the trash racks.

The bedrock at the southeastern abutment was decomposed and fractured. However, the concrete abutment appeared to extend far enough into the rock to provide an adequate cutoff.

d. Downstream Channel

Water flowing over the spillway is carried away from the dam in the natural channel of the Wallkill River. The channel is cut into bedrock and appeared to be capable of carrying all flows satisfactorily.

3.2 EVALUATION OF OBSERVATIONS

Visual observations did not reveal any serious problems which would affect the immediate safety of the dam. However, the following minor deficiencies were noted.

1. Spalling, surface cracking and separation of the gunite on portions of the downstream face of the dam.

2. The small triangular section of concrete which has been scoured away at the downstream toe.

3. The minor void under the northwest abutment.

4. The spalled concrete surfaces on the powerhouse.

SECTION 4: OPERATION AND MAINTENANCE PROCEDURES

4.1 PROCEDURE

Normal water surface is at or slightly above the spillway crest. Flow is diverted through the power house for power generation.

4.2 MAINTENANCE OF DAM

The downstream face of the dam was replaced in 1942 and 1952 using a gunite process. Maintenance of the flashboards occurs annually.

4.3 MAINTENANCE OF APPURTENANT STRUCTURES

The racks and associated equipment on the power house intake structure have been repaired or replaced several times in the past fifty years. Other maintenance has been performed on the power house as necessary.

4.4 WARNING SYSTEM IN EFFECT

No apparent warning system is in effect.

4.5 EVALUATION

Operation and maintenance of the dam is generally satisfactory; however, additional effort should be placed on maintaining the dam, to correct some of the minor deficiencies which now exist.

SECTION 5: HYDROLOGIC/HYDRAULIC

5.1 DRAINAGE AREA CHARACTERISTICS

The delineation of the contributing watershed to this dam is shown on the maps titled "Drainage Area - Dashville Dam" (Appendix D). With the drainage area encompassing some 765 square miles, the Wallkill River main stem travels approximately 89 miles from its headwaters near Lake Mohawk in the Sparta Mountains of Sussex County, New Jersey to the Dashville Dam site. Major tributaries to the Wallkill River which have been gaged (US Geological Survey streamgages) include Rutgers Creek, Pochuck Creek, Quaker Creek, and the Shawangunk Kill. Although there are no major lakes or reservoirs within the basin, an area of some 25 square miles near Pellets Island Mountain (Middletown, NY) significantly attenuates flood flows on the Wallkill because of its flat and swampy terrain. Much of the entire basin lies within the steep terrain of the Catskill Mountain area, where large population centers relative to the size of the drainage basin are minimal.

5.2 ANALYSIS CRITERIA

A limited amount of hydrologic/hydraulic information was contained in a Conservation Commission review of the application for construction (October 23, 1919) **plus** a memorandum dated 11/3/1919 summarizing the basic engineering data known about the proposed dam site. This data concerned itself with the drainage area and the maximum flood of record (Basis - 10 years of record), the design spillway capacity, and the use of flashboards on the spillway crest.

A second information source reviewed was a 1973 C.T. Main Inc. report (7) completed for the present owner. The hydrology and flood study portion (Appendix D) established a value for the standard project flood (SPF) for use in determining if the spillway discharge capacity is adequate. A review of a 1978 Phase 1 inspection report (5) for Sturgeon Pool Dam (I.D. No. NY-75), located approximately two miles downstream of Dashville Dam, established a value for the SPF peak inflow. This SPF peak inflow can be considered indicative of the routed SPF peak outflow from Dashville Dam, allowing for the increased drainage area at Sturgeon Pool Dam.

A report (4) prepared in 1977 by Water Resources Engineers, Inc., for the Corps of Engineers established values for the SPF along certain tributaries of the Lower Hudson River. The methodology described in this report employed the HEC-1 computer program in developing a model that correlated well with past known major storm events, i.e., the storms of August 17-20, 1955 and October 14-18, 1955.

The analysis of the spillway capacity of this dam was performed using streamflow gaging station records (Appendix D) and data contained in the 1977 report for the Corps of Engineers. Using the HEC-1 program, unit hydrographs were developed and routed through the Wallkill River valley and over the Dashville spillway. The spillway design flood selected for analysis was the Probable Maximum Flood (PMF; approximately twice the SPF) in accordance with the recommended guidelines of the U.S. Army Corps of Engineers.

5.3 SPILLWAY CAPACITY

The concrete gravity ogee and non-ogee sections with the flashboards act as the dam in forming the reservoir pool for the hydroelectric power station. The ogee section is 140 feet long and the non-ogee section, 215 feet long.

Discharges over the spillway were computed using weir flow relationships for the representative water surface elevations analyzed. The flashboards were designed originally, to fail when the head reached approximately 10 feet above the spillway crest. Hence, all of the analyses performed assumes no flashboards exist. Maximum theoretical discharges through the hydroelectric power station existing machinery (2 units) was determined to be 2800 cfs.

The spillway does not have sufficient capacity for discharging the peak outflow from one-half the PMF. For this storm event, the peak inflow is 68,735 cfs and the peak outflow is 68, 735 cfs, whereas the PMF peak discharge is 147,100 cfs. The computed spillway capacity is 14, 685 cfs.

5.4 RESERVOIR CAPACITY

The normal water surface is at or slightly above the spillway crest. Storage capacity for that water surface elevation is 92 acre-feet and is obtained within 2378 feet upstream of the dam. Little flood flow attenuation is achieved in the shallow-depth reservoir. With 3.5 foot high flashboards in place, the storage capacity increases to 515 acre-feet with the reservoir extending some 9 river miles upstream toward New Paltz. At the top-of-dam elevation (top of the North abutment), the storage is 965 acre-feet.

5.5 FLOODS OF RECORD

The maximum known discharge on the Wallkill River was recorded upstream at Gardiner, NY (DA of 711 sq. miles) on October 16, 1955 when a gaged flow of 30,800 cfs was measured.

5.6 OVERTOPPING POTENTIAL

Analysis indicates the spillway does not have sufficient discharge capacity for either the PMF or one-half the PMF. The computed depths of overtopping are 17.79 feet and 8.79 feet respectively. All storms exceeding approximately 10% of the PMF would result in overtopping of the dam, i.e. above the elevation of the top of the North abutment.

5.7 EVALUATION

The spillway capacity is inadequate for the peak outflow from one-half the PMF. Because of the relatively insignificant storage capacity available during these large storm events, both failure and non-failure discharges routed over the spillway attain similar water surface elevations in the downstream areas. Also, for such large storm events, high water created by the Sturgeon Pool Dam reservoir would most likely occur in the downstream areas. Therefore, the spillway capacity is considered to be inadequate since dam failure from overtopping would not significantly increase the hazard to loss of life downstream from that which would exist just before overtopping failure.

SECTION 6: STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations

Visual observations of the dam did not reveal any signs of major distress. Minor spalling and surface cracking was noted in several spots and water flowing along the downstream toe of the dam had scoured a triangular area in the concrete approximately 3 inches high and 3 inches deep. However, these deficiencies were not serious enough to affect the stability of the structure.

b. Design and Construction Data

No design computations were available concerning the structural stability of this dam. A stability analysis for this structure was performed in 1973 by Chas. T. Main of New York. The only construction records available were plans for the structure prepared by the J.G. White Engineering Corporation.

c. Data Review and Stability Evaluation

Structural and subsurface information was obtained from the 1919 construction plans prepared by the J.G. White Engineering Corporation and from the stability analysis which had been performed by Chas. T. Main.

A structural stability analysis was performed for this report since the Main study did not analyze several of the conditions which are required (ice loading, 1/2 PMF, PMF). The analysis was performed based on the cross sections of the dam shown on the plans. Analyses were made of both the high section on the southeastern end of the dam and the low section on the northwestern end. Conditions analyzed were:

1) Normal conditions with the water level at the spillway crest elevation.

2) Conditions as in 1), plus a 5000 lb/ft ice load.

3) Water level at the elevation of one-half PMF; a flow depth of 13.8 feet over the spillway crest.

4) Water level at the elevation of the PMF; a flow depth of 22.8 feet over the spillway crest.

The analyses were performed assuming full uplift at the upstream toe decreasing to a value equal to the hydrostatic pressure due to tailwater at the downstream toe.

The analyses performed (Appendix E) indicate that the factors of safety against overturning and sliding for the high section on the southeastern end of the dam are as follows:

Case - (For 38 ft. high section)	FACTORS OF SAFETY Overturning	Sliding
1) Normal Conditions	1.47	15.88
2) Ice load plus 1)	1.36	14.68
3) One-half PMF - Flow 13.8 feet over spillway	1.06	10.31
4) PMF - Flow 22.8 feet over spillway	0.93	8.40

The factors of safety against overturning and sliding for the low section on the northwestern end of the dam are as follows:

Case - (For 5 ft. high section)	FACTORS OF SAFETY Overturning	Sliding
1) Normal Conditions	1.72	266
2) Ice load plus 1)	0.78	46
3) One-half PMF - Flow 13.8 feet over spillway	1.13	52
4) PMF - Flow 22.8 feet over spillway	0.93	34

The stability analyses indicate a serious deficiency in the safety factors against overturning for both the high and low sections of the dam. The resultant force fell outside the middle third of the base for each of the conditions studied. In several cases, the resultant force acted outside the limits of the base.

The analysis which was performed for the structure under normal conditions agrees reasonably well with the Chas. T. Main analysis. The other analyses were performed for more critical conditions than those which were assumed in the Main study and as a result, the safety factors are substantially less than those listed in the Main report.

d. Post Construction Changes

The downstream face of the dam was replaced in 1942 and 1952 using a gunite process. This has been the only major post construction change.

e. Seismic Stability

This dam is located in Seismic Zone 1. Therefore, since the seismic coefficient is relatively small, a seismic stability analysis is not warranted.

SECTION 7: ASSESSMENT/RECOMMENDATIONS

7.1 ASSESSMENT

a. Safety

The Phase 1 inspection of Dashville Dam did not reveal conditions which constitute an immediate hazard to human life or property. The structure is not considered to be unsafe.

The spillway, although not having sufficient discharge capacity for passing one-half the PMF, is considered to be inadequate. A warning system should be developed and placed in readiness for future use during the occurrence of large storm events.

b. Adequacy of Information

The information available appears to be adequate for the purpose of the Phase 1 inspection except for the following:

1) The physical condition of the mass concrete beneath the gunited downstream surface of the spillway.

c. Urgency

All of the deficiencies observed during the visual inspection can be corrected during normal, continued maintenance operations.

d. Necessity for Additional Investigations

Because of the results of the Phase 1 structural stability analyses obtained for the dam, a more detailed structural stability analysis is recommended.

7.2 RECOMMENDED MEASURES

a) As a result of the detailed structural stability analysis to be completed within one year of the date of this report, remedial measures deemed necessary should be completed within two years of the date of this report.

b) Ascertain the physical condition and integrity of the mass concrete in the spillway beneath the gunited surfaces, preferably through a coring program.

c) Correct the minor deficiencies occurring on the concrete surfaces.

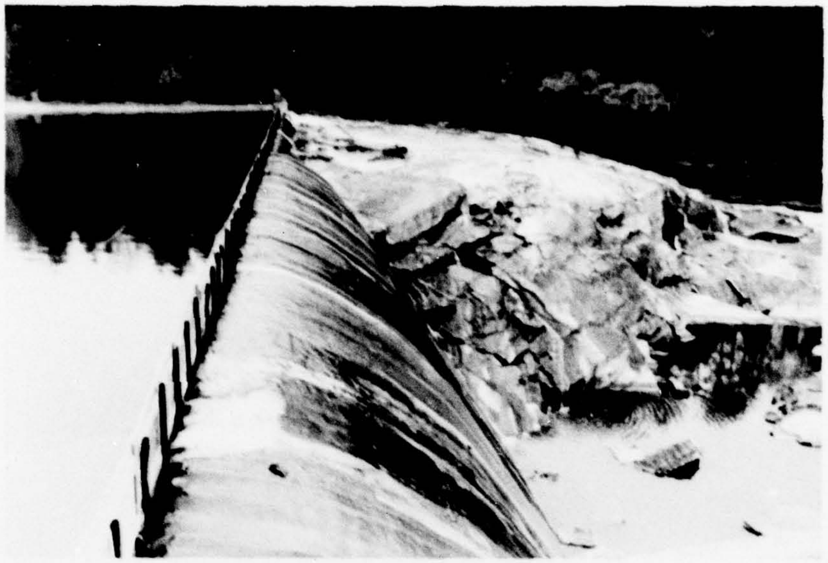
d) Develop and implement a warning system.

APPENDIX A

PHOTOGRAPHS

POWER STATION - UPSTREAM

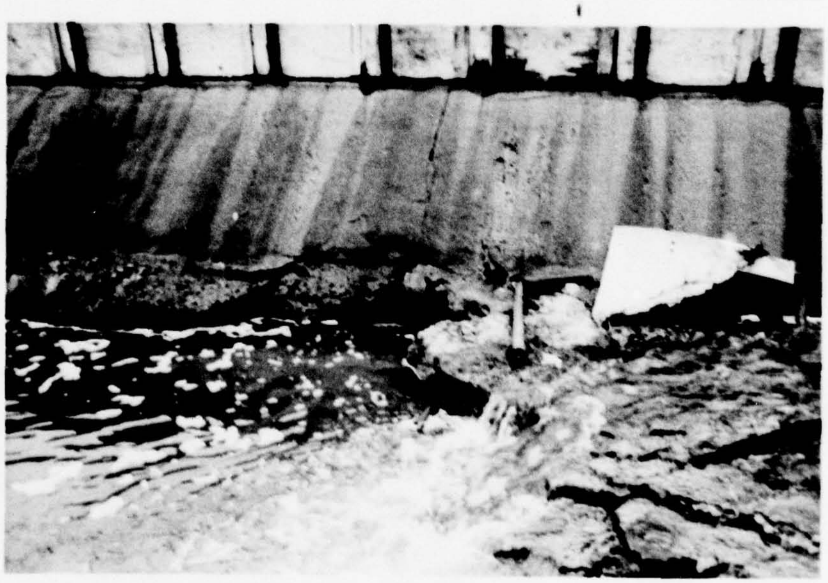
POWER STATION - DOWNSTREAM



SPILLWAY - OGEE SECTION

SPILLWAY - NON-OGEE SECTION

NORTHWEST ABUTMENT - TOP OF DAM



EROSION OF BEDROCK @ SPILLWAY

CONCRETE SURFACE SPALLING @ SPILLWAY

INTERFACE ALONG FOUNDATION AND
NON-OGEE SPILLWAY SECTION

SUBSURFACE STRATA @ POWER STATION

RESERVOIR @ DASHVILLE DAM

APPENDIX B

ENGINEERING DATA CHECKLIST

Check List
Engineering Data
Design Construction Operation

Name of Dam DASHVILLE

I.D. # NY-76
(759-LH)

Item	Remarks		
	Plans	Details	Typical Sections
Dam	YES		YES
Spillway(s)	YES		YES
Outlet(s)			
Design Reports	~~Y~~ N/A		
Design Computations	YES - 1919 CONSERVATION COMMISSION REVIEW		
Discharge Rating Curves	N/A		
Dam Stability	YES - 1973 C.T. MAIN REPORT		
Seepage Studies	N/A		
Subsurface and Materials Investigations	N/A		

Item	Remarks
Construction History	LIMITED (TO CORRESPONDENCE, 1919-1920) (APPLICATION FOR CONSTRUCTION)
Surveys, Modifications, Post-Construction Engineering Studies and Reports	N/A
Accidents or Failure of Dam Description, Reports	NONE
Operation and Maintenance Records Operation Manual	N/A

APPENDIX C

VISUAL INSPECTION CHECKLIST

VISUAL INSPECTION CHECKLIST

1) Basic Data

a. General

Name of Dam DASHVILLE DAM

I.D. # N.Y. 76 (759 L.H.)

Location: Town ESOPUS County ULSTER

Stream Name WALLKILL RIVER

Tributary of ROUNDOUT CREEK

Longitude (W), Latitude (N) W74° 2.9' N 41° 49.4'

Hazard Category SIGNIFICANT

Date(s) of Inspection 11/13/78

Weather Conditions 35° CLOUDY-OVERCAST

b. Inspection Personnel W. LYNICK R. WARRENDER

c. Persons Contacted DONALD OTIS & DICK DEMELSER - C.H. GAS & ELECTRIC

d. History:

Date Constructed 1919-1922

Owner CENTRAL HUDSON GAS & ELECTRIC

Designer J.G. WHITE ENGINEERING CORP.

Constructed by

2) Technical Data

Type of Dam CONCRETE GRAVITY W/OGEE SECTION

Drainage Area 765 SQ MILES

Height 38 Length 3~~8~~ 70

Upstream Slope VARIES Downstream Slope VARIES

4) Instrumentation

(1) Monumentation/Surveys NONE

(2) Observation Wells NONE

(3) Weirs NONE

(4) Piezometers NONE

(5) Other

5) Reservoir

a. Slopes VERTICAL IN SEVERAL SPOTS - DECOMPOSED ROCK

b. Sedimentation NONE APPARENT

6) Spillway(s) (including tail race channel)

a. General FULL LENGTH OF DAM - FLASBOARDS 3.5' HIGH ACROSS ALL BUT ONE SECTION ABOUT 10' WIDE NEAREST THE POWER HOUSE

b. Principle Spillway OVERFLOW DAM WITH FLASHBOARDS AT TIME OF INSPECTION APPROXIMATELY 1/3 OF FLASHBOARDS WERE TORNED TO ALLOW FLOW TO PASS THROUGH.

c. Emergency or Auxiliary Spillway NONE

d. Condition of ~~tail race~~ channel NATURAL BEDROCK CHANNEL- SATISFACTORY

e. Stability of Channel side/slopes SATISFACTORY

7) Downstream Channel

a. Condition (debris, etc.) NONE

b. Slopes PARTLY VERTICAL - BEDROCK

c. Approximate number of homes NONE APPARENTLY IN IMMEDIATE DANGER UNTIL THE IMPOUNDMENT FOR THE STRURGEON POND DAM

8) Miscellaneous

9) Structural

a. Concrete Surfaces SPALLING AT CORNER OF POWER HOUSE & OGEE BUTRESS WALL
2 AREAS SURFACES PATCHED FOR ENTIRE HEIGHT OF DAM

b. Structural Cracking NONE APPARENT
SOME SURFACE CRACKING WHERE MESH IS CLOSE TO SURFACE

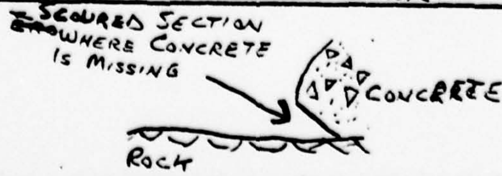
c. Movement - Horizontal & Vertical Alignment (Settlement) NO MOVEMENT EVIDENT

d. Junctions with Abutments or Embankments SOUTHWEST END- OKAY
NORTHEAST END- MINOR UNDERMINING AT END CREST OF ABUTMENT WALL CONTACT WHERE ABUTMENT RESTS ON NATURAL BEDROCK

e. Drains - Foundation, Joint, Face NONE

f. Water passages, conduits, sluices NONE

g. Seepage or Leakage NEAR MIDDLE - AREA WHERE WATER FLOWS UNDER FLASHBOARDS & THEN BENEATH THE GUNITE - IT EXITS AT THE DOWNSTREAM TOE-ROCK INTERFACE.
A TRIANGULAR SECTION OF CONCRETE 3" x 3" HAD BEEN REMOVED NEAR MID-DAM AS SHOWN IN SKETCH BELOW



h. Joints - Construction, etc. OKAY

i. Foundation ROCK - GOOD CONDITION

j. Abutments DETERIORATING ROCK

k. Control Gates ONLY GATES ARE ON POWER HOUSE

l. Approach & Outlet Channels OKAY

m. Energy Dissipators (plunge pool, etc.) - NONE - NATURAL BEDROCK CHANNEL

n. Intake Structures REPAIRS WERE BEING MADE ON TRASH RACK FOR POWER HOUSE INTAKE AT TIME OF CONSTRUCTION

o. Stability APPEARS TO BE STABLE

p. Miscellaneous

APPENDIX D

HYDROLOGIC/HYDRAULIC

ENGINEERING DATA AND COMPUTATIONS

DASHVILLE DAM
#NY-76

CHECK LIST FOR DAMS
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

AREA-CAPACITY DATA:

	Elevation (ft.)	Surface Area (acres)	Storage Capacity (acre-ft.)
1) Top of Dam (NORTH ABUTMENT)	174.0	300	965
2) Design High Water (Max. Design Pool)	NA		
3) Auxiliary Spillway Crest	NA		
4) Pool Level with Flashboards	172.5	300	515
5) Service Spillway Crest	169	9.6	92

DISCHARGES

	Volume (cfs)
1) Average Daily (PERIOD 1946 - 1961)	SUMMER - 60 WINTER - 203
2) Spillway @ Maximum High Water (ELEV. 177.0)	30,000
3) Spillway @ Design High Water	NA
4) Spillway @ Auxiliary Spillway Crest Elevation	NA
5) Low Level Outlet	NA
6) Total (of all facilities) @ Maximum High Water	
7) Maximum Known Flood	30,800
8) HYDROELECTRIC STATION EXISTING MACHINERY (2 UNITS)	2800 -(THEORETICAL MAX.)

CREST: ELEVATION: 174.0

Type: CONCRETE GRAVITY (OGEE & NON-OGEE SECTIONS)

Width: ______ Length: SPILLWAY + NORTH ABUTMENT (15')

Spillover ENTIRE LENGTH OF CREST

Location ______

SPILLWAY:

PRINCIPAL		EMERGENCY
169.0	Elevation	NONE
OGEE & NON-OGEE	Type	
— 1.5'	Width	
	Type of Control	
✓	Uncontrolled	
	Controlled:	
3.5' FLASHBOARDS	Type (Flashboards; [redacted])	
ACROSS ENTIRE CREST	Number	
355'	[redacted]/Length	
	Invert Material	
	Anticipated Length of operating service	
	Chute Length	
VARIES: 3' TO 29'	Height Between Spillway Crest & Approach Channel Invert (Weir Flow)	

OUTLET STRUCTURES/EMERGENCY DRAWDOWN FACILITIES:

Type: Gate ______ Sluice ______ Conduit ______ Penstock ______

Shape : NONE - ONLY THRU HYDROMACHINERY UNITS

Size: ______

Elevations: Entrance Invert 153.0

Exit Invert ______

Tailrace Channel: Elevation (TAILWATER) 128.0

HYDROMETEROLOGICAL GAGES:

Type : WATER-STAGE RECORDER # USGS 3715

Location: GARDINER, N.Y. 16 MILES(±) UPSTREAM

Records:

Date - SEPT 1924 TO PRESENT

Max. Reading - 30,800 cfs ON OCT. 16, 1955

FLOOD WATER CONTROL SYSTEM:

Warning System: NONE

______

Method of Controlled Releases (mechanisms):

HYDROMACHINERY UNITS

______

DRAINAGE AREA: 765 SQ MILES

DRAINAGE BASIN RUNOFF CHARACTERISTICS:

Land Use - Type: RURAL - AGRICULTURAL

Terrain - Relief: CATSKILL MOUNTAINS - STEEP

Surface - Soil: SOME EVIDENCE OF SEDIMENTATION IN RIVER BED

Runoff Potential (existing or planned extensive alterations to existing (surface or subsurface conditions)

N/A

Potential Sedimentation problem areas (natural or man-made; present or future)

N/A

Potential Backwater problem areas for levels at maximum storage capacity including surcharge storage:

N/A

Dikes - Floodwalls (overflow & non-overflow) - Low reaches along the Reservoir perimeter:

Location: NONE

Elevation: -

Reservoir:

Length @ NORMAL [redacted] Pool ELEV. 172.5 ELEV. 169 9.0 0.45 (Miles)

Length of Shoreline (@ Spillway Crest) N/A (Miles)

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
DASHVILLE DAM	1/		
SUBJECT		**COMPUTED BY**	**DATE**
DRAINAGE AREA - BELOW USGS GAGE @ GARDINER		WCL	5/15/79

USGS 7.5' QUAD : SCALE 1:24000 1" = 2000'

QUAD SHT NAME	AREA	QUAD SHT NAME	AREA
WALDEN, NY	0.30	NEWBURGH, NY	1.32
GARDINER, NY	11.10	CLINTONDALE, NY	35.77
	23.56		43.07
	33.46		16.73
			43.08
MOHONK LAKE, NY	17.96		21.73
	86.38		
		ROSENDALE, NY	43.10
			(-0.27)
			20.47
			33.64
			34.95
			293.59

Σ AREAS = 379.97 SQ INS. = 54.52 SQ MILES

1 SQ IN = 91.827 ACRES

DRAINAGE AREA @ GARDINER 711.0 SQ MILES

+ PLANIMETERED AREA 54.5

DASHVILLE DR. AREA = 765.5

SUBAREA DIVIDING LINE @ NEW PALTZ:

QUAD SHT:	AREA
CLINTONDALE, NY	
NORTH OF RTE 299	7.50
GARDINER, NY	
NORTH OF RTE 299 & WEST OF LIBERTYVILLE RD	12.20

SUBAREA 12 — AREA = 56.22 + 154.20 = 210.42 SQ INS => 30.19 SQ MI.

CORPS OF ENGR REPORT — SUBAREA 12 => 22.6 SQ MILES USE

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
DASHVILLE DAM	21		
SUBJECT		**COMPUTED BY**	**DATE**
HYDROGRAPH PARAMETERS		WCL	5/16/79

WALKILL RIVER BASIN :

1) ABOVE USGS GAGE @ GARDINER (CORPS OF ENGRS REPORT)
#3715

DR AREA = 711 SQ MILES

2) SUBAREA 12 (GAGE TO NEW PALTZ) (STA 3722.27)

DR AREA = 30.19 SQ MILES — KNOWN 3-HR HYDROGRAPH INCLUDING 1)

L = 7.27 MILES (10.65 + 8.55)

L_{CA} = 3.64 MILES

C_t = 1.7

C_P = 3.2

3) SUBAREA (NEW PALTZ TO DASHVILLE) (DAM - STA 3730.94)

DR AREA = 24.33 SQ MILES

L = 8.67 MILES (2.6 + 22.3)

L_{CA} = 4.34 MILES

C_t = 3.1

C_P = 5.0

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
DASHVILLE DAM	3/		
SUBJECT		**COMPUTED BY**	**DATE**
HYDROGRAPH PARAMETERS		WCL	5/16/79

2) SUBAREA 12 (GAGE TO NEW PALTZ) ← KNOWN 3-HR HYDROGRAPH

LAG TIME $= C_t (L \times L_{CA})^{0.3} = 4.54$ HRS t_p

UNIT RAIN DURATION $t_r = \frac{t_p}{5.5} = 0.825$ (USE 1-HR)

ADJUSTED LAG $TP = t_p + 0.25(t_R - t_r)$

$= 4.54 + 0.25(1 - 0.825)$

$TP = 4.58$ HRS

TRANSPOSITION $TF = 1 - \frac{0.3008}{(DA)^{.17718}} = 1 - \frac{0.3008}{(30.19)^{.17718}}$

$TF = 0.836$

TABLE 13 {
LOSS DATA:
INITIAL = 1.5" CONSTANT = 0.15"

BASE FLOW = 1 cfs/SQ MILE SPS RAINFALL = 9.18" TOTAL
5.08" EXCESS

QRCSN = 0.35 RTIOR = 3.0

3) SUBAREA (NEW PALTZ TO DASHVILLE)

LAG TIME = 9.20 HRS t_p

UNIT RAIN DURATION $t_r = 1.67$ (USE 1.5 HR) (OTHER UH @ 3.0 HRS)

ADJUSTED LAG $TP = 9.20 + 0.25(1.5 - 1.67)$ $TP = 9.5$ HRS ←

$TP = 9.16$ HRS

TRANSPOSITION $TF = 1 - \frac{0.3008}{(24.33)^{.17718}}$

$TF = 0.830$

					SPS RAINFALL
TABLE 13 {	LOSS DATA:	BASE FLOW:	QRCSN	RTIOR	8.57" TOTAL
	INITIAL = 1.5"	1 cfs/SQ MILE	0.35	3.0	4.62" EXCESS
	CONSTANT = 0.15"				

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
DASHVILLE DAM	4/		
SUBJECT		COMPUTED BY	DATE
HYDROGRAPH PARAMETERS		WCL	5/16/79

STA 3722.27

DRAINAGE AREA (TO NEW PALTZ) KNOWN 3-HR HYDROGRAPH

DR. AREA = 741.2 SQ MILES (711 + 30.2)

L = 30.07 MILES (NODES $\underbrace{101-108}_{39.2}$ + 7.27) + 33.6)

L_{CA} = 40.04 MILES

C_t = 1.46 $\quad C_t = \frac{t_p}{(L \times L_{CA})^{0.3}} \quad t_p = t_r \times 5.5 = 16.5$

CP = 16.03 $\quad$ WTD CP = $\frac{\Sigma A \times CP}{\Sigma A}$

TP = 16.5

CHANNEL ROUTING OF THIS HYDROGRAPH TO DAM:

DISTANCE = 8.67 MILES

(ASSUME) VELOCITY = 8 fps $\quad V = \frac{1.486}{n} r^{2/3} S^{1/2}$

∴ TRAVEL TIME ≈ 2.16 HRS $\quad V = 0.495\, r^{2/3}$

CLINTONDALE QUAD:

180 CONTOUR - 16500'

(DIST. TO RTE 299) - 8.25"

+8.67 MILES - 45800'

\# OF ROUTING STEPS = $\frac{2.16}{3} \approx 1$

CREST ELEV. = 169

USE: 200' x 10' MAIN CHANNEL

2000' TOTAL WIDTH

SLOPE = $\frac{\Delta h}{L} = \frac{11}{62300} = 0.0001766$

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
DASHVILLE DAM	5/		
SUBJECT		**COMPUTED BY**	**DATE**
STAGE - STORAGE DATA		WCL	5/17/79

39.4% OGEE SECTION ≈ 140'

60.6% GRAVITY SECTION ≈ 215'

L = 355' @ CREST ELEV. 169.0

NORTH ABUTMENT - +5' @ ELEV. 174.0

SOUTH ABUTMENT - +12' @ POWER STATION @ ELEV. 177.0

	STAGE		AREA	ΔS	(AC-FT) STORAGE
SOUTH ABUT.	177	PROJECTED	300		1865.7
				900	
NORTH ABUT.	174		300		965.7
				450	
FLSH-BDS	172.5	3.5'	300 ACRES		515.66
				423.6	
CREST	169		9.55		92.06

← 300 ACRES = 9 MILES × ω

275' = ω_{AVE}

175' × 2378' = 9.55 ACRES

$$\Delta V = \frac{\Delta h}{3}\left(A_1 + A_2 + \sqrt{A_1 A_2}\right)$$

$$\Delta S = \frac{3.5}{3}\left(9.55 + 300 + \sqrt{(9.55)(300)}\right) = 423.6$$

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
DASHVILLE DAM	6/		
SUBJECT		**COMPUTED BY**	**DATE**
SPILLWAY DISCHARGE DATA		WCL	5/17/79

39.4% OGEE SECTION — L = 140'

60.6% GRAVITY SECTION — L = 215'

DESIGNER DATA: 10/1919

@ 50000 cfs H = 10' => L = 370' => ELEV 179 ± C = 3.5 ←

BUREC - DESIGN OF SMALL DAMS (1977)

FIG 249 - OGEE SECTION

$P = 29'$ FOR $H_o < 9.7'$ $C_o > 3.95$

ENTIRE DAM - C = 3.7 ←

(FOR HEC-1 ANALYSIS) L = 370'

C = 3.7 $Q = CLH^{3/2}$

STAGE	H_1 (L=355')	H_2 (L=15')	Q_1	Q_2	TOTAL Q	
SOUTH ABUT. 177.0			29721	288	30009	
	3	3				
NORTH ABUT. 174.0			14685	—	14685	TOP-OF-DAM ←
	5					
CREST 169.0						

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
DASHVILLE DAM	7/		
SUBJECT: SPILLWAY DISCHARGE w/ FLASHBOARDS		COMPUTED BY: WCL	DATE: 5/17/79

3.5' FLASHBOARDS

SHARP-CRESTED WEIR

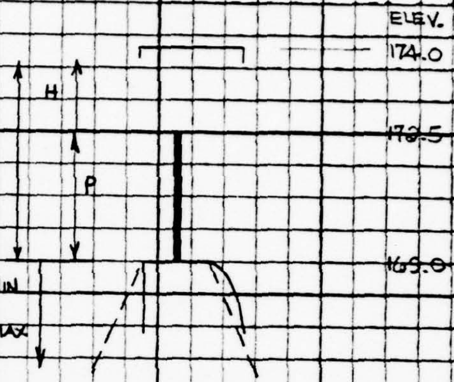
$L/b = 1$

$Q = CLH^{3/2}$

$Q = (3.44)(355)(1.5)^{3/2} = 2243$ cfs

$L = 215'$ 5' MIN

$L = 140'$ 29' MAX



CONDITION: WATER SURFACE @ TOP OF DAM

$H = 1.5'$ $d = 5'$ $P = 3.5'$

FRANCIS FORMULA: $(v < 5.0 \text{ fps})$ $C = 3.33\left(1 + 0.259\frac{H^2}{d^2}\right)$ $C = 3.408$ 1)

$(v < 1.0 \text{ fps})$ $C = 3.33\left[\left(1+\frac{h}{H}\right)^{3/2} - \left(\frac{h}{H}\right)^{3/2}\right]$ $C = 3.378$ 2)

$h = \frac{v^2}{2g}$ for $v = 1$ $h = 0.0155$

FTELEY & STEARNS: $(v < 2.0 \text{ fps})$ $C = 3.31\left(1 + 0.383\frac{H^2}{d^2} + \frac{0.007}{H^{3/2}}\right)$ $C = 3.437$ 3)

BAZIN FORMULA: $C = \left(3.248 + \frac{0.079}{H}\right)\left(1 + 0.55\frac{H^2}{d^2}\right)$ $C = 3.464$ 4)

FRESE FORMULA: $C = \left(3.288 + \frac{0.0368}{H}\right)\left(1 + 0.55\frac{H^2}{d^2}\right)$ $C = 3.477$ 5)

KING: $C = \frac{3.34}{H^{0.03}}\left(1 + 0.56\frac{H^2}{d^2}\right)$ $C = 3.466$ 6)

REHBOCK FORMULA: $C = 3.235 + \frac{1}{60H - 0.56} + 0.428\frac{H}{P}$ $C = 3.430$ 7)

SWISS SOCIETY FORMULA: $C = \left(3.288 + \frac{1}{92.8H - 0.49}\right)\left(1 + 0.5\frac{H^2}{d^2}\right)$ $C = 3.443$ 8)

AVE. VALUE:

1) - 8) $C = 3.438$ EXCL 2) 3) $C = 3.448$ USE $C = 3.44$

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
GASHVILLE DAM	8/		
SUBJECT		**COMPUTED BY**	**DATE**
DISCHARGE CAPACITY - HYDROMACHINERY UNITS		WCL	5/17/79

2 EXISTING UNITS - RATED @ 3850 HP EACH

MAXIMUM THEORETICAL DISCHARGE CAPACITY (100% EFFICIENT):

CREST ELEV. 169

℄ OF UNIT ELEV. 145

24'

$HP = \frac{Qh}{8.8}$ OR $Q = \frac{HP(8.8)}{h}$

$= \frac{3850(8.8)}{24}$

$Q = 1411.7$ cfs

USE $Q_{max} \approx 2800$ cfs

Dashville Dam NY-76

Dam Breaching - Downstream Flood Wave Analysis

1) HEC-1 DB requires the breached section to not be located at an overflow section. At this site, the entire dam is an overflow section.

2) Initial analysis using suggested breach parameters resulted in large errors between the computed and interpolated breach hydrographs.

3) Because of the extremely short time intervals used in the breach hydrograph generation and the relatively long storm unit hydrograph time interval, the TFAIL variable selected exceeded the upper limit of the suggested parameter for a concrete gravity dam failure time of 0.5 hours.

4) Since both Q (spillway) and Q (breach) are assumed to occur independently at the site in HEC-1 DB analysis, the BRWID value was selected on the smaller side to minimize duplication of the discharges.

1

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*******************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
  LAST MODIFICATION  26 FEB 79
  MODIFIED FOR HONEYWELL APR 79
*******************************

**************************************
THIS PROGRAM IS CURRENTLY BEING MODIFIED
TO RUN ON THE DGS HONEYWELL SYSTEM

PLEASE REPORT ANY UNUSUAL OPERATING PROBLEMS
TO MIKE TILLSON (RM. 423) PH: 7-5666
**************************************
      1              A DASHVILLE DAM    NY-76    WALLKILL RIVER                 LOWER HUDSON RIVER BASIN
      2              A                           ULSTER COUNTY                  FLOOD WAVE ANALYSIS
      3              A                           SPF PARAMETERS - CORPS OF ENGRS REPORT(LHRBHFRM)
      4              B     100        3       0       0       0       0       0       0       0       0
      5              B1      5
      6              J       1        2       1
      7              J1      1        2
      8              K       0   722.27                                       1
      9              K1               3-HOUR RUNOFF HYDROGRAPH DATA - NEW PALTZ
     10              M      -1        0   741.2           741.2   0.907                           1
     11              N    1623     1619    1605    1585    1563    1539    1514    1485    1451    1412
     12              N    1368     1320    1269    1252    1437    1949    2742    3458    3634    3509
     13              N    5319    13935   30474   52554   71839   79948   76423   67189   58499   51609
     14              N   46615    43042   40659   39794   39191   38619   37972   37201   36288   35244
     15              N   34101    32889
     16              N       0
     17              N       0
     18              N       0
     19              N       0
     20              N       0
     21              K       1   730.94                                       1
     22              K1               CHANNEL ROUTING OF HYDROGRAPH TO DAM
     23              Y                                1       1
     24              Y1      1                                             -169
     25              Y6    .05      .04     .03     169     200   62300 .000176
     26              Y7      0      200      40     180     900     179     900     169    1100     169
```

	K	0	730.94					1			
	K1		INFLOW HYDROGRAPH TO DAM - SUBAREA (NEW PALTZ TO DAM)								
	M	1	1	24.3		24.3	0.83			1	
SPF PRECIP.	P	8.57									
	T							1.5	0.15		
	W	9.5	5								
	X	24	69	3							
	K	2	730.94					1			
	K1		COMBINED HYDROGRAPHS AT DAM								
	K	1	730.94					1			
	K1		COMBINED HYDROGRAPH ROUTED OVER DAM - RECTANGULAR BREACH								
	Y				1	1					
	Y1	1						-169			
	$S	0	92.06	515.7	965.7	1865.7					
	$E	140	169	172.5	174	177					
	$$	169	355	3.7	1.5						
	$D	174	3.7	1.5	15						
	$B	50	0	164	3.0	169	179				
	K	1	731.9					1			
	K1		DOWNSTREAM OF DAM					100	FT		
	Y				1	1					
	Y1	1									
	Y6	.04	.04	.04	135	180	100	.34			
	Y7	80	180	140	140	350	136	390	125	400	135
	Y7	450	136	460	140	520	180				
	K	1	771.9					1			
	K1		RIVER CHANNEL EXPANSION					4100	FT		
	Y				1	1					
	Y1	1									
	Y6	.04	.04	.04	131.1	180	4000	.000975			
	Y7	80	180	240	160	300	140	350	131.1	500	131.1
	Y7	520	140	560	160	700	180				

```
            Y7  1100     179    1960     180    2000     200
            K      0  730.94                                         1
            K1                INFLOW HYDROGRAPH TO DAM - SUBAREA (NEW PALTZ TO DAM)
            M      1       1    24.3            24.3    0.83                         1
SPF PRECIP. P   8.57
            T                                                       1.5     0.15
            W    9.5       5
            X     24      69       3
            K      2  730.94                                         1
            K1                COMBINED HYDROGRAPHS AT DAM
            K      1  730.94                                         1
            K1                COMBINED HYDROGRAPH ROUTED OVER DAM - NO BREACH          9A
            Y                                  1       1
            Y1     1                                               -169
            $S     0   92.06   515.7   965.7  1865.7
            $E   140     169   172.5     174     177
            $$   169     355     3.7     1.5
            $D   174     3.7     1.5      15
            K     99
            A
            A
            A
            A
            A
```

```
K       1   800.9   .                               1
K1              STURGEON POOL RESERVOIR             6000    FT
Y                               1       1
Y1      1
Y6    .04     .04     .04     131     180     2900.0000345
Y7     50     180     180     160     410     140     490     131     1280     131
Y7   1310     140    1380     160    2190     180
K      99
A
A
A
A
A
```

```
*******************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
  LAST MODIFICATION  26 FEB 79
  MODIFIED FOR HONEYWELL APR 79
*******************************

*************************************
THIS PROGRAM IS CURRENTLY BEING MODIFIED
TO RUN ON THE DGS HONEYWELL SYSTEM

PLEASE REPORT ANY UNUSUAL OPERATING PROBLEMS
TO MIKE TILLSON (RM. 423) PH: 7-5666
*************************************

RUN  DATE 06/05/79
                         DASHVILLE DAM   NY-76   WALLKILL RIVER                LOWER HUDSON RIVER BASIN
                                                 ULSTER COUNTY                 FLOOD WAVE ANALYSIS
                                                 SPF PARAMETERS - CORPS OF ENGRS REPORT(LHRBHFRM)

                                                        JOB SPECIFICATION
                         NQ       NHR      NMIN      IDAY       IHR      IMIN     METRC      IPLT      IPRT     NSTAN
                        100         3         0         0         0         0         0         0         0         0
                                                    JOPER       NWT     LROPT     TRACE
                                                        5         0         0         0

                                          MULTI-PLAN ANALYSES TO BE PERFORMED
                                                NPLAN= 1 NRTIO= 2 LRTIO= 1
                           RTIOS=    1.00    2.00
                                    1/2PMF    PMF

                 *********          *********          *********          *********          *********

                                                SUB-AREA RUNOFF COMPUTATION

                                    3-HOUR RUNOFF HYDROGRAPH DATA - NEW PALTZ
                                      ISTAQ     ICOMP     IECON     ITAPE      JPLT      JPRT     INAME    ISTAGE     IAUTO
                                     722.27         0         0         0         0         0         1         0         0

                                                        HYDROGRAPH DATA
                         IHYDG      IUHG     TAREA      SNAP     TRSDA     TRSPC     RATIO     ISNOW     ISAME     LOCAL
                            -1         0    741.20        0.    741.20      0.91        0.         0         1         0

                                                        INPUT HYDROGRAPH
                 1623.      1619.      1605.      1585.      1563.      1539.      1514.      1485.      1451.      1412.
                 1363.      1320.      1269.      1252.      1437.      1949.      2742.      3458.      3634.      3509.
                 5319.     13935.     30474.     52554.     71839.     79948.     76423.     67189.     58499.     51609.
                46515.     43042.     40659.     39784.     39191.     38619.     37972.     37201.     36288.     35244.
                34101.     32889.         0.         0.         0.         0.         0.         0.         0.         0.
                    0.         0.         0.         0.         0.         0.         0.         0.         0.         0.
                    0.         0.         0.         0.         0.         0.         0.         0.         0.         0.
                    0.         0.         0.         0.         0.         0.         0.         0.         0.         0.
                    0.         0.         0.         0.         0.         0.         0.         0.         0.         0.
                    0.         0.         0.         0.         0.         0.         0.         0.         0.         0.

                                           PEAK    6-HOUR    24-HOUR    72-HOUR   TOTAL VOLUME
                                  CFS    79948.    77040.     62490.     40613.       1005917.
                                  CMS     2264.     2182.      1770.      1150.         28484.
                               INCHES                0.97       3.14       6.12           6.31
```

THOUS CU M	47121.	152887.	298091.	307631.

HYDROGRAPH AT STA722.27 FOR PLAN 1, RTIO 1

1623.	1619.	1605.	1583.	1563.	1539.	1314.	1485.	1451.	1412.
1368.	1320.	1269.	1252.	1437.	1949.	2742.	3458.	3634.	3309.
5319.	13733.	30474.	52554.	71839.	79948.	76423.	67189.	58499.	51609.
46615.	43042.	40659.	39784.	39191.	38619.	37972.	37201.	36283.	35244.
34101.	32839.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	79948.	77040.	62490.	40613.	1005917.
CMS	2264.	2182.	1770.	1150.	28484.
INCHES		0.97	3.14	6.12	6.31
MM		24.56	79.68	155.36	160.33
AC-FT		38201.	123947.	241666.	249401.
THOUS CU M		47121.	152887.	298091.	307631.

HYDROGRAPH AT STA722.27 FOR PLAN 1, RTIO 2

3246.	3238.	3210.	3170.	3126.	3078.	3028.	2970.	2902.	2824.
2736.	2640.	2538.	2504.	2874.	3898.	5484.	6916.	7268.	7018.
10638.	27870.	60948.	105108.	143678.	159896.	152846.	134378.	116998.	103218.
93230.	86084.	81318.	79563.	78382.	77238.	75944.	74402.	72576.	70488.
68202.	65778.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	159896.	154079.	124980.	81227.	2011833.
CMS	4528.	4363.	3539.	2300.	56969.
INCHES		1.93	6.27	12.23	12.62
MM		49.12	159.36	310.72	320.67
AC-FT		76403.	247894.	483332.	498802.
THOUS CU M		94241.	305773.	596182.	615263.

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HYDROGRAPH ROUTING

CHANNEL ROUTING OF HYDROGRAPH TO DAM

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
730.94	1	0	0	0	0	1	0	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME	IOPT	IPMP	LSTR
0.	0.	0.	1	1	0	0	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.	0.	0.	-169.	0

NORMAL DEPTH CHANNEL ROUTING

6

QN(1)	QN(2)	QN(3)	ELNVT	ELMAX	RLNTH	SEL
0.0500	0.0400	0.0500	169.0	200.0	62300.	0.00018

CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC

0.	200.00	40.00	180.00	900.00	179.00	900.00	169.00	1100.00	169.00
1100.00	179.00	1960.00	180.00	2000.00	200.00				

STORAGE	0.	466.70	933.40	1400.10	1866.80	2333.50	2800.20	5533.16	10025.02	14532.12
	19054.45	23592.00	28144.79	32712.80	37296.05	41894.52	46508.22	51137.15	55781.31	60440.70
OUTFLOW	0.	221.09	694.51	1350.91	2159.67	3101.01	4160.42	5964.98	9952.27	15554.44
	22527.27	30734.52	40084.08	50507.61	61951.51	74372.12	87732.89	102002.58	117154.11	133163.61
STAGE	169.00	170.63	172.26	173.89	175.53	177.16	178.79	180.42	182.05	183.68
	185.32	186.95	188.58	190.21	191.84	193.47	195.11	196.74	198.37	200.00
FLOW	0.	221.09	694.51	1350.91	2159.67	3101.01	4160.42	5964.98	9952.27	15554.44
	22527.27	30734.52	40084.08	50507.61	61951.51	74372.12	87732.89	102002.58	117154.11	133163.61

STATION 730.94, PLAN 1, RTIO 1

OUTFLOW

180.	460.	725.	984.	1159.	1273.	1350.	1403.	1426.	1428.
1414.	1390.	1356.	1327.	1332.	1456.	1771.	2251.	2769.	3090.
3671.	4893.	7993.	16408.	32486.	51004.	63970.	67901.	65364.	60293.
55007.	50216.	46527.	43746.	41868.	40561.	39604.	38785.	37957.	37068.
36096.	35040.	27809.	18230.	12913.	9590.	7689.	6164.	5184.	4400.
2939.	1809.	1193.	843.	618.	480.	372.	289.	225.	198.
176.	156.	139.	124.	110.	98.	87.	77.	69.	61.
54.	48.	43.	38.	34.	30.	27.	24.	21.	19.
17.	15.	13.	12.	10.	9.	8.	7.	7.	6.
5.	5.	4.	4.	3.	3.	3.	2.	2.	2.

STOR

380.	703.	955.	1139.	1264.	1346.	1399.	1430.	1443.	1444.
1437.	1422.	1403.	1383.	1387.	1461.	1642.	1912.	2169.	2328.
2584.	3910.	7818.	15085.	24445.	32911.	38043.	39499.	38559.	36632.
34515.	32585.	30968.	29750.	28927.	28354.	27911.	27512.	27109.	26676.
26203.	25689.	21975.	16267.	12407.	9617.	7475.	5758.	4351.	3163.
2253.	1664.	1292.	1039.	856.	722.	616.	534.	470.	418.
371.	330.	294.	261.	232.	206.	183.	163.	145.	129.
115.	102.	91.	81.	72.	64.	57.	50.	45.	40.
35.	31.	28.	25.	22.	20.	17.	16.	14.	12.
11.	10.	9.	8.	7.	6.	5.	5.	4.	4.

STAGE

170.3	171.5	172.3	173.0	173.4	173.7	173.9	174.0	174.0	174.0
174.0	174.0	173.9	173.8	173.8	174.1	174.7	175.7	176.6	177.1
178.0	179.5	181.3	183.9	187.3	190.3	192.1	192.6	192.3	191.6
190.9	190.2	189.6	189.2	188.9	188.7	188.5	188.4	188.2	188.1
187.9	187.7	186.4	184.3	182.9	181.9	181.1	180.5	179.7	179.0
176.9	174.8	173.5	172.6	172.0	171.5	171.2	170.9	170.6	170.5
170.3	170.2	170.0	169.9	169.8	169.7	169.6	169.6	169.5	169.5
169.4	169.4	169.3	169.3	169.3	169.2	169.2	169.2	169.2	169.1
169.1	169.1	169.1	169.1	169.1	169.1	169.1	169.1	169.0	169.0
169.0	169.0	169.0	169.0	169.0	169.0	169.0	169.0	169.0	169.0

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	67901.	66284.	57082.	39012.	1007434.
CMS	1923.	1877.	1616.	1105.	28527.
INCHES		0.83	2.87	5.88	6.32
MM		21.13	72.79	149.23	160.57
AC-FT		32568.	113220.	232137.	249777.

MAXIMUM STORAGE = 39499.

MAXIMUM STAGE IS 192.6

7

STATION 730.94, PLAN 1, RTIO 2

OUTFLOW

501.	1251.	1930.	2404.	2702.	2862.	2938.	2963.	2952.	2916.
2862.	2792.	2711.	2635.	2657.	2948.	3699.	4380.	4790.	5146.
5703.	8309.	17747.	42264.	82018.	121240.	142228.	143055.	132680.	119195.
107043.	97228.	89748.	84742.	81695.	79643.	78030.	76521.	74919.	73192.
71262.	69119.	51578.	29767.	19317.	13527.	9924.	7957.	6379.	5316.
4511.	3198.	1939.	1269.	892.	644.	500.	388.	302.	234.
202.	179.	159.	142.	126.	112.	100.	89.	79.	70.
62.	55.	49.	44.	39.	35.	31.	27.	24.	22.
19.	17.	15.	13.	12.	11.	9.	8.	7.	7.
6.	5.	5.	4.	4.	3.	3.	3.	2.	2.

STOR

743.	1329.	1734.	1988.	2135.	2215.	2253.	2265.	2260.	2242.
2215.	2180.	2140.	2102.	2113.	2258.	2597.	3133.	3754.	4293.
5137.	8174.	15954.	29100.	44535.	56971.	63079.	63319.	60300.	56375.
52682.	49588.	47162.	45475.	44423.	43714.	43158.	42636.	42083.	41458.
40743.	39950.	33141.	23057.	16972.	12901.	9994.	7777.	6000.	4550.
3332.	2376.	1739.	1342.	1074.	883.	742.	632.	546.	480.
426.	378.	336.	299.	266.	236.	210.	187.	166.	148.
131.	117.	104.	92.	82.	73.	65.	58.	51.	46.
41.	36.	32.	28.	25.	23.	20.	18.	16.	14.
13.	11.	10.	9.	8.	7.	6.	5.	5.	4.

STAGE

171.6	173.6	175.1	175.9	176.5	176.7	176.9	176.9	176.9	176.8
176.7	176.6	176.5	176.4	176.4	176.9	178.1	179.0	179.4	179.7
180.2	181.4	184.2	188.9	194.4	198.8	200.9	201.0	200.0	198.6
197.3	196.2	195.3	194.7	194.4	194.1	193.9	193.7	193.5	193.3
193.1	192.8	190.4	186.8	184.6	183.1	182.0	181.2	180.6	179.8
179.1	177.3	175.1	173.7	172.8	172.1	171.6	171.2	170.9	170.7
170.5	170.3	170.2	170.0	169.9	169.8	169.7	169.7	169.6	169.5
169.5	169.4	169.4	169.3	169.3	169.3	169.2	169.2	169.2	169.2
169.1	169.1	169.1	169.1	169.1	169.1	169.1	169.1	169.1	169.0
169.0	169.0	169.0	169.0	169.0	169.0	169.0	169.0	169.0	169.0

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	143055.	140254.	118569.	78965.	2014811.
CMS	4051.	3972.	3357.	2236.	57053.
INCHES		1.76	5.95	11.89	12.64
MM		44.71	151.19	302.07	321.14
AC-FT		69548.	235178.	469872.	499540.
THOUS CU M		85786.	290088.	579578.	616173.

MAXIMUM STORAGE = 63319.

MAXIMUM STAGE IS 201.0

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SUB-AREA RUNOFF COMPUTATION

INFLOW HYDROGRAPH TO DAM - SUBAREA (NEW PALTZ TO DAM)

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
730.94	0	0	0	0	0	1	0	0

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
1	1	24.30	0.	24.30	0.83	0.	0	1	0

PRECIP DATA

SPFE	PMS	R6	R12	R24	R48	R72	R96
8.57	0.	0.	0.	0.	0.	0.	0.

LOSS DATA

LROPT	STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0	0.	0.	1.00	0.	0.	1.00	1.50	0.15	0.	0.

UNIT HYDROGRAPH DATA

TP= 9.50 CP=5.00 NTA= 0

RECESSION DATA

STRTQ= 24.00 QRCSN= 69.00 RTIOR= 3.00

CLARK DID NOT CONVERGE TO GIVEN SNYDER COEFFICIENTS
APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC= 5.70 AND R= 0.50 INTERVALS

UNIT HYDROGRAPH 7 END-OF-PERIOD ORDINATES, LAG= 9.50 HOURS, CP= 0.78 VOL= 1.00

271. 768. 1136. 1242. 1045. 602. 159.

0

END-OF-PERIOD FLOW

MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q	MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
1.01	3.00	1	0.00	0.	0.00	22.	1.07	9.00	51	0.	0.	0.	6.
1.01	6.00	2	0.00	0.	0.00	19.	1.07	12.00	52	0.	0.	0.	5.
1.01	9.00	3	0.01	0.	0.01	17.	1.07	15.00	53	0.	0.	0.	5.
1.01	12.00	4	0.01	0.	0.01	15.	1.07	18.00	54	0.	0.	0.	4.
1.01	15.00	5	0.07	0.	0.07	14.	1.07	21.00	55	0.	0.	0.	4.
1.01	18.00	6	0.14	0.	0.14	12.	1.08	0.	56	0.	0.	0.	4.
1.01	21.00	7	0.01	0.	0.01	11.	1.08	3.00	57	0.	0.	0.	3.
1.02	0.	8	0.01	0.	0.01	10.	1.08	6.00	58	0.	0.	0.	3.
1.02	3.00	9	0.01	0.	0.01	9.	1.08	9.00	59	0.	0.	0.	3.
1.02	6.00	10	0.01	0.	0.01	8.	1.08	12.00	60	0.	0.	0.	2.
1.02	9.00	11	0.05	0.	0.05	7.	1.08	15.00	61	0.	0.	0.	2.
1.02	12.00	12	0.05	0.	0.05	6.	1.08	18.00	62	0.	0.	0.	2.
1.02	15.00	13	0.31	0.	0.31	6.	1.08	21.00	63	0.	0.	0.	2.
1.02	18.00	14	0.62	0.	0.62	5.	1.09	0.	64	0.	0.	0.	1.
1.02	21.00	15	0.03	0.	0.03	5.	1.09	3.00	65	0.	0.	0.	1.
1.03	0.	16	0.03	0.	0.03	4.	1.09	6.00	66	0.	0.	0.	1.
1.03	3.00	17	0.07	0.	0.07	4.	1.09	9.00	67	0.	0.	0.	1.
1.03	6.00	18	0.07	0.	0.07	3.	1.09	12.00	68	0.	0.	0.	1.
1.03	9.00	19	0.37	0.	0.37	3.	1.09	15.00	69	0.	0.	0.	1.
1.03	12.00	20	0.37	0.	0.37	3.	1.09	18.00	70	0.	0.	0.	1.
1.03	15.00	21	2.29	1.84	0.45	501.	1.09	21.00	71	0.	0.	0.	1.
1.03	18.00	22	4.65	4.20	0.45	2552.	1.10	0.	72	0.	0.	0.	1.
1.03	21.00	23	0.20	0.	0.20	5311.	1.10	3.00	73	0.	0.	0.	1.
1.04	0.	24	0.20	0.	0.20	7053.	1.10	6.00	74	0.	0.	0.	0.
1.04	3.00	25	0.00	0.	0.00	7137.	1.10	9.00	75	0.	0.	0.	0.
1.04	6.00	26	0.00	0.	0.00	5495.	1.10	12.00	76	0.	0.	0.	0.
1.04	9.00	27	0.02	0.	0.02	2821.	1.10	15.00	77	0.	0.	0.	0.
1.04	12.00	28	0.02	0.	0.02	670.	1.10	18.00	78	0.	0.	0.	0.
1.04	15.00	29	0.12	0.	0.12	68.	1.10	21.00	79	0.	0.	0.	0.
1.04	18.00	30	0.24	0.	0.24	61.	1.11	0.	80	0.	0.	0.	0.
1.04	21.00	31	0.01	0.	0.01	55.	1.11	3.00	81	0.	0.	0.	0.
1.05	0.	32	0.01	0.	0.01	49.	1.11	6.00	82	0.	0.	0.	0.
1.05	3.00	33	0.	0.	0.	44.	1.11	9.00	83	0.	0.	0.	0.
1.05	6.00	34	0.	0.	0.	39.	1.11	12.00	84	0.	0.	0.	0.
1.05	9.00	35	0.	0.	0.	35.	1.11	15.00	85	0.	0.	0.	0.
1.05	12.00	36	0.	0.	0.	32.	1.11	18.00	86	0.	0.	0.	0.
1.05	15.00	37	0.	0.	0.	28.	1.11	21.00	87	0.	0.	0.	0.
1.05	18.00	38	0.	0.	0.	25.	1.12	0.	88	0.	0.	0.	0.
1.05	21.00	39	0.	0.	0.	23.	1.12	3.00	89	0.	0.	0.	0.
1.06	0.	40	0.	0.	0.	20.	1.12	6.00	90	0.	0.	0.	0.

1.06	6.00	42	0.	0.	0.	16.	1.12	12.00	92	0.	0.	0.	0.
1.06	9.00	43	0.	0.	0.	15.	1.12	15.00	93	0.	0.	0.	0.
1.06	12.00	44	0.	0.	0.	13.	1.12	18.00	94	0.	0.	0.	0.
1.06	15.00	45	0.	0.	0.	12.	1.12	21.00	95	0.	0.	0.	0.
1.06	18.00	46	0.	0.	0.	11.	1.13	0.	96	0.	0.	0.	0.
1.06	21.00	47	0.	0.	0.	9.	1.13	3.00	97	0.	0.	0.	0.
1.07	0.	48	0.	0.	0.	8.	1.13	6.00	98	0.	0.	0.	0.
1.07	3.00	49	0.	0.	0.	8.	1.13	9.00	99	0.	0.	0.	0.
1.07	6.00	50	0.	0.	0.	7.	1.13	12.00	100	0.	0.	0.	0.
									SUM	9.99	6.03	3.96	32374.
										(254.)(	153.)(	100.)(	916.73)

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	7137.	6705.	3915.	1337.	32369.
CMS	202.	190.	111.	38.	917.
INCHES		2.57	6.00	6.14	6.20
MM		65.20	152.28	155.95	157.37
AC-FT		3325.	7766.	7953.	8025.
THOUS CU M		4101.	9579.	9810.	9899.

HYDROGRAPH AT STA730.94 FOR PLAN 1, RTIO 1

22.	19.	17.	15.	14.	12.	11.	10.	9.	8.
7.	6.	6.	5.	5.	4.	4.	3.	3.	3.
501.	2552.	5311.	7053.	7137.	5495.	2821.	670.	68.	61.
55.	49.	44.	39.	35.	32.	28.	25.	23.	20.
18.	16.	15.	13.	12.	11.	9.	8.	8.	7.
6.	5.	5.	4.	4.	4.	3.	3.	3.	2.
2.	2.	2.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	7137.	6705.	3915.	1337.	32369.
CMS	202.	190.	111.	38.	917.
INCHES		2.57	6.00	6.14	6.20
MM		65.20	152.28	155.95	157.37
AC-FT		3325.	7766.	7953.	8025.
THOUS CU M		4101.	9579.	9810.	9899.

HYDROGRAPH AT STA730.94 FOR PLAN 1, RTIO 2

43.	39.	35.	31.	28.	25.	22.	20.	18.	16.
14.	13.	12.	10.	9.	8.	7.	7.	6.	5.
1003.	5104.	10622.	14105.	14274.	10990.	5642.	1340.	137.	122.
110.	98.	88.	79.	71.	63.	57.	51.	46.	41.
37.	33.	29.	26.	24.	21.	19.	17.	15.	14.
12.	11.	10.	9.	8.	7.	6.	6.	5.	5.
4.	4.	3.	3.	3.	2.	2.	2.	2.	2.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	14274.	13411.	7831.	2673.	64737.
CMS	404.	380.	222.	76.	1833.
INCHES		5.13	11.99	12.28	12.39
MM		130.40	304.57	311.90	314.73
AC-FT		6650.	15532.	15906.	16051.
THOUS CU M		8203.	19159.	19620.	19798.

********** ********** ********** ********** **********

COMBINE HYDROGRAPHS

COMBINED HYDROGRAPHS AT DAM

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
730.94	2	0	0	0	0	1	0	0

SUM OF 2 HYDROGRAPHS AT 730.94 PLAN 1 RTIO 1

202.	480.	743.	999.	1173.	1288.	1361.	1413.	1435.	1436.
1422.	1396.	1362.	1332.	1337.	1460.	1774.	2255.	2772.	3093.
4172.	7445.	13304.	23460.	39623.	56499.	66791.	68571.	65432.	60354.
55062.	50266.	46571.	43766.	41903.	40593.	39632.	38810.	37980.	37088.
36114.	35057.	27824.	18243.	12924.	9601.	7698.	6173.	5192.	4407.
2945.	1814.	1203.	847.	622.	483.	376.	292.	227.	200.
178.	158.	141.	125.	111.	99.	88.	78.	70.	62.
55.	49.	43.	39.	34.	31.	27.	24.	21.	19.
17.	15.	13.	12.	11.	9.	8.	7.	7.	6.
5.	5.	4.	4.	3.	3.	3.	2.	2.	2.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	68571.	67341.	58259.	40184.	1039803.
CMS	1942.	1907.	1650.	1138.	29444.
INCHES		0.82	2.83	5.86	6.32
MM		20.79	71.93	148.84	160.47
AC-FT		33392.	115555.	239110.	257802.
THOUS CU M		41189.	142535.	294938.	317995.

SUM OF 2 HYDROGRAPHS AT 730.94 PLAN 1 RTIO 2

544.	1290.	1964.	2435.	2729.	2887.	2961.	2982.	2970.	2932.
2876.	2805.	2722.	2645.	2666.	2957.	3706.	4387.	4796.	5152.
6706.	13413.	28369.	56370.	96291.	132230.	147870.	144394.	132816.	119317.
107153.	97326.	89836.	84821.	81766.	79706.	78087.	76572.	74965.	73233.
71299.	69152.	51607.	29794.	19341.	13548.	9943.	7974.	6394.	5329.
4524.	3209.	1949.	1278.	900.	651.	506.	394.	307.	239.
206.	183.	163.	145.	129.	114.	102.	90.	80.	71.
64.	57.	50.	45.	40.	35.	31.	28.	25.	22.
20.	17.	16.	14.	12.	11.	10.	9.	8.	7.
6.	5.	5.	4.	4.	3.	3.	3.	2.	2.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	147870.	143091.	121771.	81491.	2079548.
CMS	4187.	4052.	3448.	2308.	58886.
INCHES		1.74	5.92	11.88	12.64
MM		44.17	150.34	301.84	320.94
AC-FT		70954.	241530.	484908.	515591.
THOUS CU M		87521.	297923.	598125.	635972.

********** ********** ********** ********** **********

HYDROGRAPH ROUTING

COMBINED HYDROGRAPH ROUTED OVER DAM - RECTANGULAR BREACH

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
730.94	1	0	0	0	0	1	0	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME	IOPT	IPMP	LSTR
0.	0.	0.	1	1	0	0	0

1 0 0 0. 0. 0. -169. 0

CAPACITY=	0.	92.	516.	966.	1866.
ELEVATION=	140.	169.	173.	174.	177.

CREL	SPWID	COQW	EXPW	ELEVL	COQL	CAREA	EXPL
169.0	355.0	3.7	1.5	0.	0.	0.	0.

DAM DATA

TOPEL	COQD	EXPD	DAMWID
174.0	3.7	1.5	15.

DAM BREACH DATA

BRWID	Z	ELBM	TFAIL	WSEL	FAILEL
50.	0.	164.00	3.00	169.00	179.00

STATION 730.94, PLAN 1, RATIO 1

BEGIN DAM FAILURE AT 78.00 HOURS

END-OF-PERIOD HYDROGRAPH ORDINATES

OUTFLOW

162.	354.	639.	926.	1134.	1261.	1344.	1401.	1431.	1437.
1426.	1403.	1371.	1339.	1333.	1420.	1689.	2142.	2671.	3050.
3908.	6805.	11587.	20454.	35904.	53729.	66605.	68259.	66327.	61072.
55960.	50938.	47140.	44221.	42183.	40805.	39787.	38957.	38134.	37257.
36301.	35260.	29755.	20338.	13977.	9966.	7867.	6409.	5293.	4549.
3273.	2067.	1251.	824.	655.	463.	401.	276.	245.	188.
190.	151.	149.	121.	116.	96.	92.	77.	72.	61.
57.	49.	44.	39.	35.	31.	28.	24.	22.	19.
18.	15.	14.	12.	11.	10.	9.	8.	7.	6.
5.	5.	4.	4.	3.	3.	3.	2.	2.	2.

STORAGE

122.	143.	163.	188.	202.	210.	215.	218.	220.	221.
220.	219.	217.	215.	214.	220.	235.	260.	286.	304.
342.	454.	747.	1332.	2166.	2970.	3073.	3134.	3062.	2852.
2662.	2461.	2301.	2176.	2088.	2027.	1982.	1944.	1907.	1867.
1823.	1775.	1510.	1011.	621.	445.	379.	328.	287.	255.
193.	126.	89.	86.	85.	83.	82.	81.	81.	80.
80.	79.	79.	79.	79.	79.	78.	78.	78.	78.
78.	78.	78.	77.	77.	77.	77.	77.	77.	77.
77.	77.	77.	77.	77.	77.	77.	77.	77.	77.
77.	77.	76.	76.	76.	76.	76.	76.	76.	76.

STAGE

169.2	169.4	169.6	169.8	169.9	170.0	170.0	170.0	170.1	170.1
170.1	170.0	170.0	170.0	170.0	170.1	170.2	170.4	170.6	170.8
171.1	172.0	173.3	175.2	178.0	180.7	181.0	181.2	181.0	180.3
179.7	179.0	178.5	178.0	177.7	177.5	177.4	177.3	177.1	177.0
176.9	176.7	175.8	174.2	172.9	171.9	171.4	171.0	170.6	170.4
169.9	169.3	168.0	167.1	166.6	166.1	165.9	165.5	165.4	165.1
165.1	165.0	165.0	164.8	164.8	164.7	164.7	164.6	164.6	164.5
164.5	164.5	164.4	164.4	164.4	164.3	164.3	164.3	164.3	164.2
164.2	164.2	164.2	164.2	164.2	164.2	164.1	164.1	164.1	164.1
164.1	164.1	164.1	164.1	164.1	164.1	164.1	164.1	164.1	164.1

PEAK OUTFLOW IS 68259. AT TIME 84.00 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	68259.	67363.	58166.	40141.	1038928.
CMS	1933.	1907.	1647.	1137.	29419.
INCHES		0.82	2.83	5.85	6.31

92.8H - 0.49 d²

C = 3.443

AVE. VALUE:
1) - 3) C = 3.438
EXCL 2) 3) C = 3.448
USE C = 3.44

OVN

STATION 730.94, PLAN 1, RATIO 2

BEGIN DAM FAILURE AT 72.00 HOURS

END-OF-PERIOD HYDROGRAPH ORDINATES

OUTFLOW

545.	1012.	1840.	2345.	2684.	2863.	2950.	2981.	2974.	2941.
2888.	2820.	2740.	2660.	2654.	2881.	3539.	4293.	4742.	5094.
6375.	11384.	23924.	50370.	92128.	127378.	147873.	145047.	134618.	120854.
108678.	98461.	90766.	85373.	82123.	79961.	78305.	76789.	75205.	73492.
71597.	69481.	55577.	33519.	20371.	14917.	10385.	8104.	6660.	5430.
4669.	3488.	2236.	1382.	927.	730.	448.	452.	264.	280.
175.	210.	143.	162.	116.	126.	94.	97.	76.	76.
61.	59.	49.	46.	39.	37.	31.	29.	25.	23.
20.	18.	15.	14.	13.	11.	10.	9.	8.	7.
6.	6.	5.	4.	4.	4.	3.	3.	3.	2.

STORAGE

159.	194.	244.	270.	287.	296.	300.	301.	301.	299.
297.	293.	290.	286.	285.	296.	326.	359.	377.	391.
439.	732.	1534.	2829.	3975.	5093.	5694.	5613.	5309.	4895.
4315.	4186.	3930.	3746.	3633.	3557.	3499.	3445.	3388.	3326.
3257.	3180.	2647.	1693.	1041.	682.	457.	386.	337.	292.
261.	209.	139.	90.	86.	85.	83.	83.	81.	81.
80.	80.	79.	79.	79.	79.	78.	79.	78.	78.
78.	78.	78.	78.	77.	77.	77.	77.	77.	77.
77.	77.	77.	77.	77.	77.	77.	77.	77.	77.
77.	77.	77.	76.	76.	76.	76.	76.	76.	76.

STAGE

169.6	169.8	170.3	170.5	170.6	170.7	170.7	170.7	170.7	170.7
170.7	170.7	170.6	170.6	170.6	170.7	170.9	171.2	171.4	171.5
171.9	173.2	175.9	180.2	184.0	187.8	189.8	189.5	188.5	187.1
185.8	184.7	183.9	183.3	182.9	182.6	182.4	182.3	182.1	181.9
181.6	181.4	179.6	176.4	174.3	173.1	172.0	171.4	171.0	170.7
170.4	170.0	169.4	168.3	167.1	166.8	166.0	166.1	165.4	165.5
165.1	165.2	164.9	165.0	164.8	164.9	164.7	164.7	164.6	164.6
164.5	164.5	164.5	164.4	164.4	164.4	164.3	164.3	164.3	164.3
164.3	164.2	164.2	164.2	164.2	164.2	164.2	164.2	164.1	164.1
164.1	164.1	164.1	164.1	164.1	164.1	164.1	164.1	164.1	164.1

PEAK OUTFLOW IS 147873. AT TIME 81.00 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	147873.	143146.	121794.	81481.	2079433.
CMS	4187.	4053.	3449.	2307.	58883.
INCHES		1.74	5.92	11.88	12.63
MM		44.18	150.37	301.80	320.92
AC-FT		70982.	241576.	484844.	515562.
THOUS CU M		87554.	297979.	598047.	635936.

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FORMULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

				½ PMF	PMF
				RATIOS APPLIED TO FLOWS	
OPERATION	STATION	AREA	PLAN	RATIO 1 1.00	RATIO 2 2.00
HYDROGRAPH AT	722.27 (	741.20 0.00)	1	79948. (2263.88)(	159896. 4527.75)(
ROUTED TO	730.94 (	741.20 0.00)	1	67901. (1922.75)(	143055. 4050.86)(
HYDROGRAPH AT	730.94 (	24.30 0.00)	1	7137. (202.10)(	14274. 404.19)(
2 COMBINED	730.94 (	765.50 0.00)	1	68571. (1941.72)(	147870. 4187.21)(
ROUTED TO	730.94 (	765.50 0.00)	1	68259. (1932.87)(	147873. 4187.28)(
ROUTED TO	731.9 (	765.50 0.00)	1	68103. (1928.47)(	148400. 4202.22)(
ROUTED TO	771.9 (	765.50 0.00)	1	68378. (1936.26)(	147455. 4175.47)(
ROUTED TO	800.9 (	765.50 0.00)	1	68726. (1946.10)(	146799. 4156.87)(

PLAN 1 STATION 730.94

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	67901.	192.6	84.00
2.00	143055.	201.0	84.00

13 A

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FORMULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

NO BREACH

				½PMF	PMF	
OPERATION	STATION	AREA	PLAN	RATIO 1 1.00	RATIO 2 2.00	RATIOS APPLIED TO FLOWS
HYDROGRAPH AT	722.27	741.20 (0.00)	1	79948. (2263.88)(	159396. 4527.75)(	
ROUTED TO	730.94	741.20 (0.00)	1	67901. (1922.75)(	143055. 4050.86)(	
HYDROGRAPH AT	730.94	24.30 (0.00)	1	7137. (202.10)(	14274. 404.19)(	
2 COMBINED	730.94	765.50 (0.00)	1	68571. (1941.72)(	147870. 4187.21)(	
ROUTED TO	730.94	765.50 (0.00)	1	68735. (1946.35)(	147098. 4165.34)(	

PLAN 1 STATION 730.94

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	67901.	192.6	84.00
2.00	143055.	201.0	84.00

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	169.00	169.00	174.00
STORAGE	92.	92.	966.
OUTFLOW	0.	0.	14685.

	RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
½ PMF	1.00	181.23	7.23	3134.	68259.	63.00	84.00	78.00
PMF	2.00	189.76	15.76	5694.	147873.	69.00	81.00	72.00

PLAN 1 STATION 731.9

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	68103.	140.9	84.00
2.00	148400.	143.3	81.00

PLAN 1 STATION 771.9

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	68376.	159.9	84.00
2.00	147455.	173.6	81.00

PLAN 1 STATION 800.9

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	68726.	163.7	84.00
2.00	146799.	179.1	84.00

14A

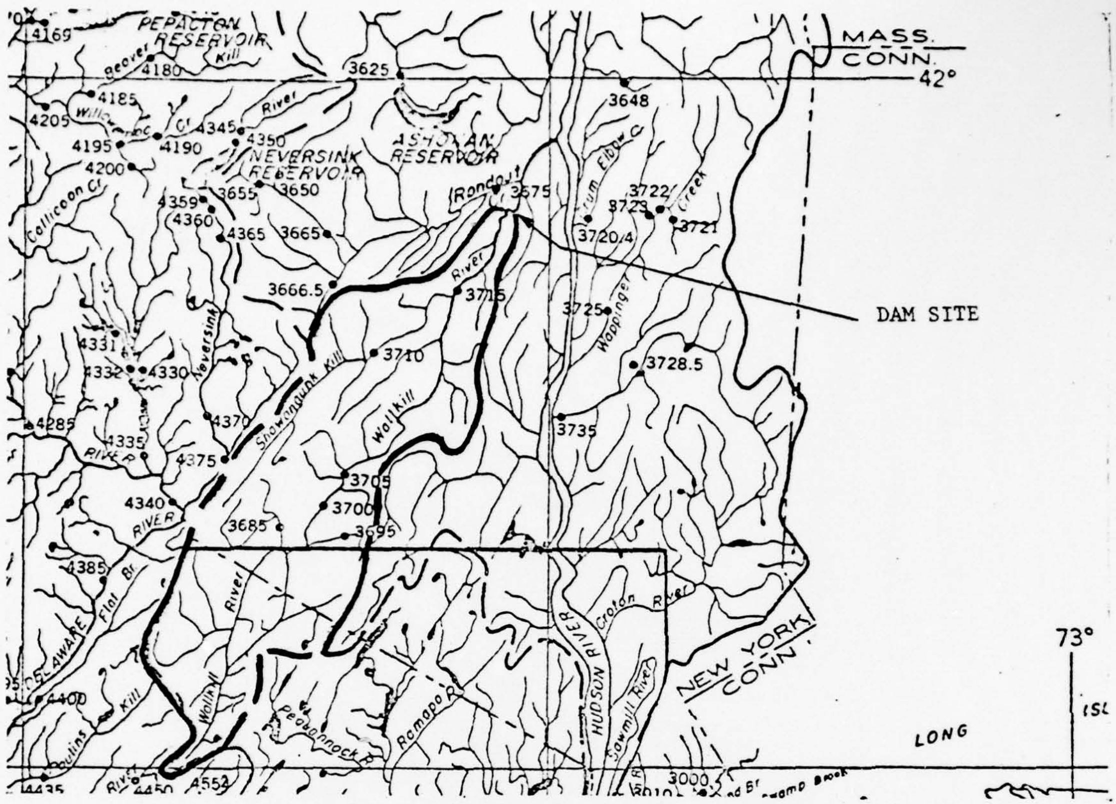
SUMMARY OF DAM SAFETY ANALYSIS

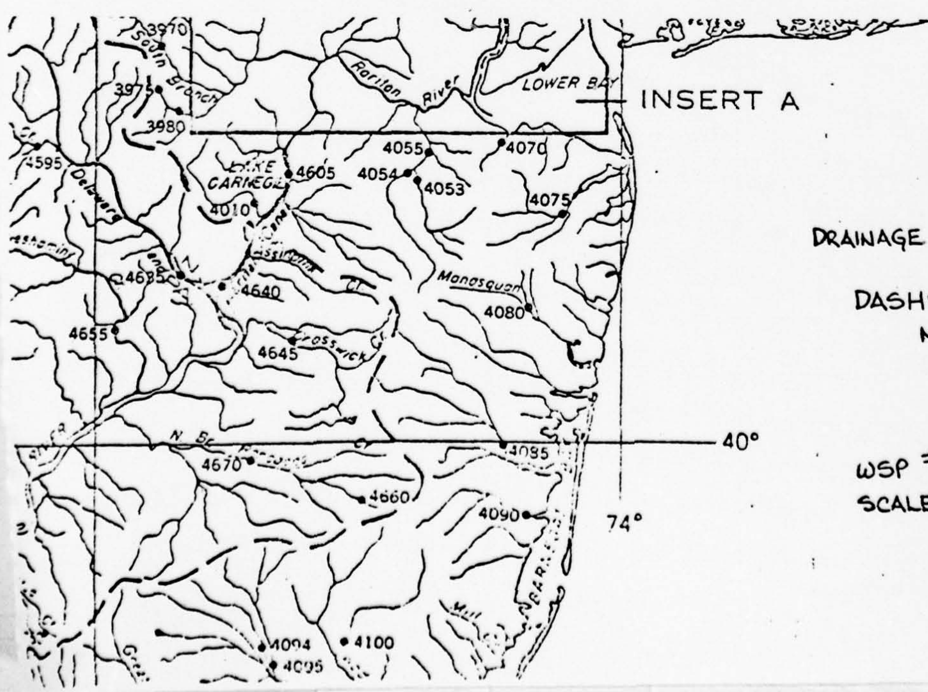
NO BREACH

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	169.00	169.00	174.00
STORAGE	92.	92.	966.
OUTFLOW	0.	0.	14685.

	RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
½ PMF	1.00	182.79	8.79	3604.	68735.	63.00	84.00	0.
PMF	2.00	191.79	17.79	6304.	147098.	72.00	81.00	0.

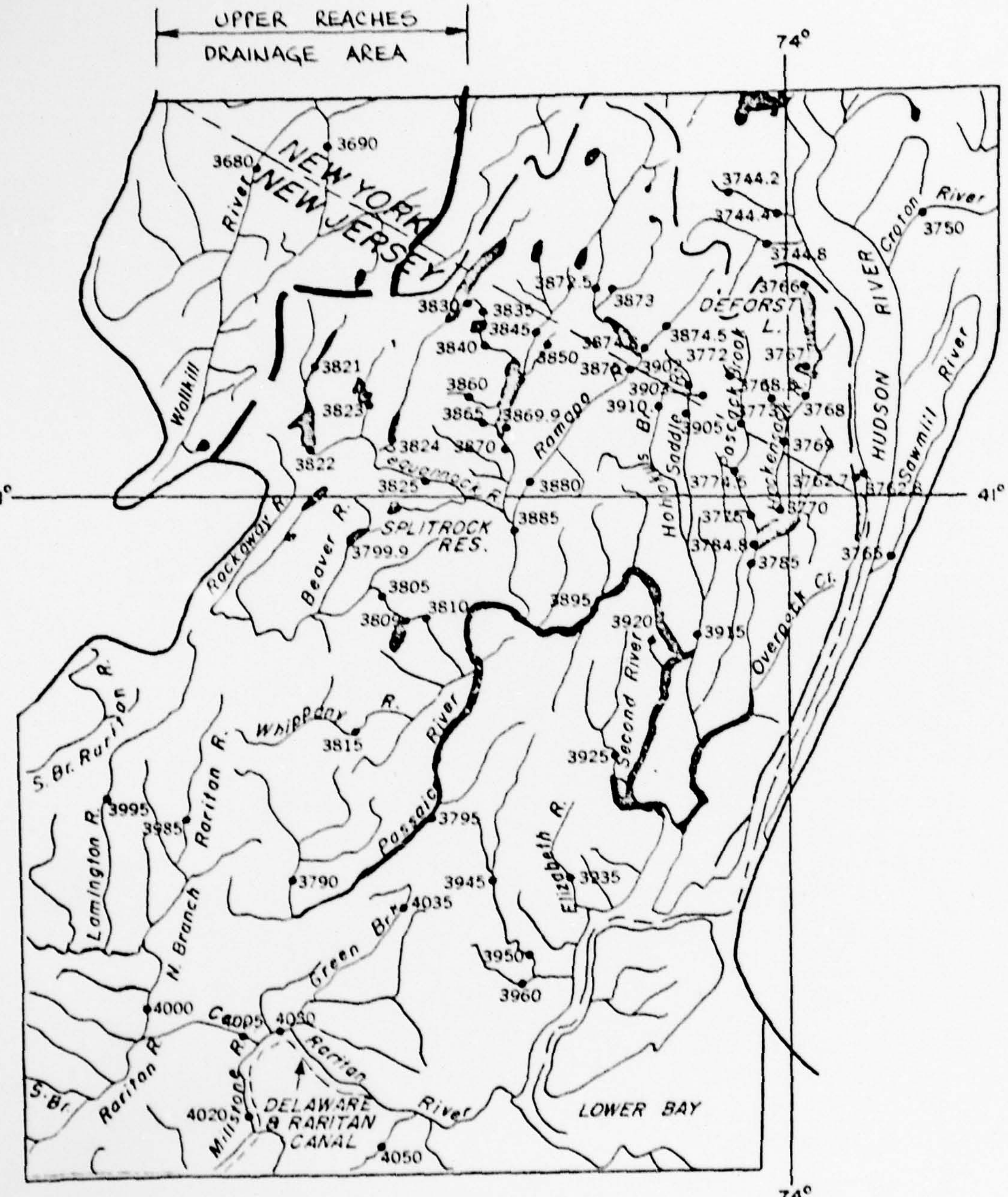




DRAINAGE AREA

DASHVILLE DAM
NY-76

WSP #1722
SCALE: 1:1,000,000



INSERT A

DRAINAGE AREA

DASHVILLE DAM

NY - 76

WSP # 1722

WALLKILL RIVER/SPILLWAY FLOODS

Results of previous analyses:

1) Original Design (1919): 10 feet of overtopping @ 50,000 cfs

2) C.T. Main report (1973): SPF inflow - 62,000 cfs
SPF routed outflow - 38,000 cfs

Unit hydrograph derived from a September, 1960 storm

3) Corps of Engineers report (1977):

(computer simulation @ New Paltz) SPF - 79948 cfs

Comparison of known recorded floods vs. simulated floods:

@ Gardiner:

	known recorded floods	simulated floods
August 1955	30,300 cfs	30,255 cfs
October 1955	30,800 cfs	30,620 cfs

4) Phase I inspection report (1978) for Sturgeon Pool Dam:

SPF inflow - 85,800 cfs

Yearly discharge, in cubic feet per second, of Shawangunk Kill at Pine Bush, N. Y.

Year	W.S.P. no.	Water year ending Sept. 30							Calendar year	
		Momentary maximum		Minimum day	Mean	Per square mile	Runoff in inches		Mean	Runoff in inches
		Discharge	Date							
1925	601	*2,240	Mar. 28, 1925	10	94.7	0.928	12.60		119	15.85
1926	621	*2,070	Nov. 16, 1925	8	140	1.37	18.65		143	18.99
1927	641,1502	*7,350	Sept. 1, 1927	16	*222	*2.18	*29.51		*297	*39.58
1928	661,1502	*4,100	Nov. 3, 1927	34	*307	*3.01	*40.94		*204	*27.20
1929	681,1502	*3,800	Mar. 5, 1929	9.2	*133	*1.30	*17.71		*152	*20.24
1930	696	*1,910	Mar. 6, 1930	3.5	113	1.11	15.05		96.1	12.80
1931	711	*2,240	Mar. 29, 1931	14	129	1.26	17.11		125	16.67
1932	726	1,110	Apr. 1, 1932	7.6	92.0	.902	12.28		-	-

* Revised.

122. Wallkill River at Gardiner, N. Y.

Location.--Lat 41°41'10", long 74°09'55", on left bank 400 ft upstream from highway bridge, 500 ft downstream from Shawangunk Kill, and three-quarters of a mile northwest of Gardiner, Ulster County.

Drainage area.--711 sq mi.

Gage.--Water-stage recorder. Datum of gage is 185.70 ft above mean sea level, adjustment of 1912.

Average discharge.--26 years (1924-50), 1,028 cfs.

Extremes.--1924-50: Maximum discharge, 21,000 cfs Dec. 31, 1948 (gage height, 14.81 ft); maximum gage height, 18.83 ft Mar. 7, 1945 (ice jam); minimum discharge, 20 cfs Sept. 17, 1932; minimum gage height, 1.94 ft Sept. 11, 1944.

Remarks.--Large diurnal fluctuation during low and medium flow caused by powerplant above station.

Monthly and yearly mean discharge, in cubic feet per second

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1925	352	138	344	102	2,490	2,290	856	565	235	375	338	293	687
1926	289	1,120	1,160	718	1,180	2,910	1,270	311	283	176	685	376	872
1927	574	1,330	717	836	2,090	2,980	784	1,380	440	217	997	2,180	1,210
1928	1,920	3,410	2,660	1,030	1,970	1,390	2,150	1,110	1,760	2,740	1,730	966	1,900
1929	227	256	316	601	1,140	2,930	2,440	1,100	360	176	96.5	125	812
1930	496	757	1,050	951	1,040	1,890	925	428	486	224	136	365	728
1931	107	632	307	624	1,240	2,320	1,690	1,200	1,480	925	282	210	916
1932	108	122	378	1,142	1,320	1,050	1,970	682	519	178	103	51.4	631
1933	579	3,010	764	1,030	1,180	2,510	2,870	631	323	118	1,150	2,290	1,360
1934	535	399	732	1,734	404	1,963	2,766	1,162	497	291	144	1,682	1,032
1935	1,045	904	1,321	1,120	1,211	2,195	1,126	497	334	555	126	189	885
1936	157	1,496	991	1,517	483	5,947	2,261	611	327	112	146	110	1,184
1937	398	391	1,704	2,675	1,855	1,411	1,674	1,511	646	450	311	441	1,120
1938	330	1,367	1,246	1,731	1,609	1,741	1,292	625	756	2,306	1,177	2,664	1,364
1939	886	675	2,715	1,179	2,548	2,735	2,978	598	190	111	105	92.2	1,235
1940	262	787	349	270	414	2,992	4,322	1,178	1,131	384	181	767	1,083
1941	251	1,447	1,610	928	1,418	1,614	1,211	239	343	383	168	54.8	800
1942	62.4	167	569	712	1,060	2,541	919	536	483	518	1,547	880	833
1943	1,316	1,984	2,064	1,497	1,978	2,150	961	1,504	716	215	94.8	72.1	1,209
1944	451	1,530	758	398	913	2,059	2,750	697	320	110	60.5	126	763
1945	81.5	462	1,161	912	986	3,593	1,154	1,939	1,324	2,430	1,431	1,458	1,427
1946	775	1,355	2,014	1,926	548	1,890	465	1,155	1,093	380	197	133	1,000
1947	186	148	157	689	579	1,898	1,958	2,350	787	635	447	161	857
1948	90.3	1,477	536	418	1,321	3,726	2,102	1,644	1,073	609	233	68.1	1,106
1949	83.0	351	1,212	3,429	1,927	970	954	936	191	144	92.9	76.5	863
1950	94.7	134	497	990	1,120	2,983	1,295	1,258	1,140	474	222	226	861

Monthly and yearly runoff, in inches

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1925	0.57	0.22	0.56	0.17	3.65	3.71	1.34	0.92	0.37	0.61	0.55	0.46	13.13
1926	.47	1.75	1.89	1.16	1.73	4.71	2.00	.50	.44	.29	1.11	.59	16.64
1927	.93	2.08	1.16	1.44	3.07	4.83	1.23	2.24	.69	.35	1.62	3.42	23.06
1928	3.11	5.35	4.31	1.67	2.99	2.26	3.37	1.81	2.76	4.44	2.80	1.52	36.39
1929	.37	.40	.50	.97	1.68	4.78	3.83	1.78	.57	.29	.16	.20	15.53
1930	.80	1.19	1.71	1.54	1.52	3.07	1.45	.69	.76	.36	.22	.57	13.88
1931	.17	.99	.50	1.03	1.82	3.76	2.66	1.95	2.33	1.50	.46	.33	17.50
1932	.17	.19	.61	1.85	2.00	1.71	3.09	1.11	.81	.29	.17	.08	12.08
1933	.94	4.72	1.24	1.66	1.73	4.07	4.51	1.02	.51	.19	1.86	3.59	26.04
1934	.95	.63	1.19	2.81	.59	3.18	4.34	1.88	.78	.47	.23	2.64	19.69
1935	1.70	1.42	2.14	1.82	1.77	3.56	1.76	.81	.52	.90	.20	.30	16.90
1936	.25	2.34	1.60	2.46	.73	9.64	3.55	.99	.51	.18	.24	.17	22.66
1937	.65	.61	2.77	4.34	2.72	2.28	2.62	2.46	1.01	.73	.50	.69	21.38
1938	.63	2.14	2.02	2.80	2.35	2.02	2.03	1.03	1.18	3.74	1.91	4.18	26.03
1939	1.44	1.37	4.40	1.91	3.73	4.44	4.57	.97	.30	.18	.17	.14	23.57
1940	.42	1.24	.57	.44	.65	4.85	6.78	1.91	1.77	.62	.29	1.20	20.72

Monthly and yearly runoff, in in[ches]

Water year	Oct.	Nov.	Dec.	Jan.	Feb.
1941	0.41	2.26	2.61	1.51	[illegible]
1942	.10	.25	.92	1.15	[illegible]
1943	2.13	3.11	3.35	2.43	[illegible]
1944	.73	2.40	.47	.65	[illegible]
1945	.13	.72	1.88	1.48	[illegible]
1946	1.26	2.13	3.27	[illegible]	[illegible]
1947	.30	.23	.26	1.44	[illegible]
1948	.15	2.32	.87	[illegible]	[illegible]
1949	.13	.55	1.96	5.56	[illegible]
1950	.15	.21	.81	[illegible]	[illegible]

Yearly dis[charge]

Year	W.S.P. no.	Water [year]	
		Momentary maxim[um]	
		Discharge	Date
1925	601	#8,500	Feb. 12, 1925
1926	621	#5,500	Mar. 3, 1926
1927	641	12,900	Sept. 2, 1927
1928	661	11,400	Nov. 5, 1927
1929	681	9,450	Mar. 6, 1929
1930	696	5,970	Sept. 17, 1930
1931	711	6,990	June 17, 1931
1932	726	7,300	Apr. 1, 1932
1933	741	10,600	Nov. 22, 1932
1934	756	8,840	Sept. 17, 1934
1935	781	6,550	Dec. 1, 1934
1936	801	18,000	Mar. 12, 1936
1937	821	8,500	May 15, 1937
1938	851	17,500	Sept. 22, 1938
1939	871	11,700	Dec. 6, 1938
1940	891	13,700	Mar. 31, 1940
1941	921	6,600	Dec. 29, 1940
1942	951	6,380	Mar. 9, 1942
1943	971	13,000	Dec. 30, 1942
1944	1001	9,320	Nov. 9, 1943
1945	1051	13,900	July 23, 1945
1946	1051	7,330	May 28, 1946
1947	1081	8,100	Apr. 6, 1947
1948	1111	13,700	Mar. 17, 1948
1949	1141	21,000	Dec. 31, 1948
1950	1171	7,690	Mar. 9, 1950

\# Not previously published.

123. Wall[kill River ...]

Location.--Lat 41°44'50", long 74°[...] Paltz, Ulster County.

Drainage area.--739 sq mi (revised).

Gage.--Chain gage. Altitude of ga[ge ...]

Monthly and yearly [...]

Water year	Oct.	Nov.	Dec.	Jan.	Feb.
1901	-	-	-	-	-
1902	528	517	3,325	2,139	[illegible]
1903	2,076	704	2,611	1,729	[illegible]
1904	4,286	-	-	-	-

† Corrected.
\# Not previously published; partly es[...]

Monthly [...]

Water year	Oct.	Nov.	Dec.	Jan.	Feb.
1901	-	-	-	-	-
1902	0.82	0.78	5.18	3.34	[illegible]
1903	3.24	1.06	4.07	[illegible]	[illegible]
1904	6.69	-	-	-	-

† Corrected.
\# Not previously published; partly e[...]

Yearly di[scharge]

Year	W.S.P. no.	Water [year]	
		Momentary maxim[um]	
		Discharge	Date
1902	97	-	-
1903	97	-	-

\# Not previously published.

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Monthly and yearly runoff, in inches, of Wallkill River at Gardiner, N. Y.--Continued

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1941	0.41	2.26	2.61	1.51	2.07	2.62	1.90	0.39	0.54	0.62	0.27	0.09	15.29
1942	.10	.25	.92	1.15	1.55	4.12	1.44	.87	.76	.84	2.50	1.38	15.68
1943	2.13	3.11	3.35	2.43	2.90	3.49	1.51	2.44	1.12	.35	.15	.11	23.09
1944	.73	2.40	.47	.65	1.39	3.34	3.53	1.13	.50	.18	.10	.20	14.62
1945	.13	.72	1.88	1.48	1.44	6.31	1.81	3.14	1.76	3.94	2.32	2.29	27.22
1946	1.26	2.13	3.27	3.12	.80	3.06	.73	1.87	1.72	.62	.32	.21	19.11
1947	.30	.23	.26	1.44	.65	3.08	3.12	3.81	1.23	1.03	.73	.28	16.36
1948	.15	2.32	.87	.68	2.00	6.04	3.30	2.67	1.68	.99	.38	.11	21.19
1949	.13	.55	1.90	5.56	2.82	1.57	1.54	1.52	.30	.23	.15	.12	16.45
1950	.15	.21	.81	1.44	1.64	4.84	2.03	2.04	1.79	.77	.36	.35	16.43

Yearly discharge, in cubic feet per second

Year	W.S.P. no.	Water year ending Sept. 30						Calendar year	
		Momentary maximum		Minimum day	Mean	Per square mile	Runoff in inches	Mean	Runoff in inches
		Discharge	Date						
1925	601	#8,500	Feb. 12, 1925	52	687	0.966	13.13	832	15.89
1926	621	#5,500	Mar. 3, 1926	50	872	1.23	16.64	875	16.70
1927	641	12,900	Sept. 2, 1927	135	1,210	1.70	23.06	1,660	31.66
1928	661	11,400	Nov. 3, 1927	234	1,900	2.67	36.39	1,300	24.69
1929	681	9,450	Mar. 6, 1929	36	812	1.14	15.93	940	17.96
1930	696	5,970	Sept.17, 1930	36	728	1.02	13.88	621	11.84
1931	711	6,990	June 17, 1931	70	916	1.29	17.50	881	16.81
1932	726	7,300	Apr. 1, 1932	23	631	.887	12.08	940	18.01
1933	741	10,600	Nov. 20, 1932	29	1,360	1.91	26.04	1,148	21.91
1934	756	8,840	Sept.17, 1934	55	1,032	1.45	19.69	1,162	22.18
1935	781	6,550	Dec. 1, 1934	67	885	1.24	16.90	830	15.83
1936	801	18,000	Mar. 12, 1936	55	1,184	1.67	22.66	1,175	22.50
1937	821	8,500	May 15, 1937	94	1,120	1.58	21.38	1,160	22.14
1938	851	17,500	Sept.22, 1938	99	1,364	1.92	26.03	1,491	28.45
1939	871	11,200	Dec. 6, 1938	41	1,235	1.74	23.57	973	18.59
1940	891	13,700	Mar. 31, 1940	62	1,083	1.52	20.72	1,243	23.77
1941	921	6,600	Dec. 29, 1940	33	800	1.13	15.29	591	11.28
1942	951	6,380	Mar. 9, 1942	34	833	1.17	15.88	1,216	23.20
1943	971	13,000	Dec. 30, 1942	44	1,209	1.70	23.09	945	18.10
1944	1001	9,320	Nov. 9, 1943	76	763	1.07	14.62	718	13.75
1945	1031	13,900	July 23, 1945	38	1,427	2.01	27.22	1,632	31.15
1946	1051	7,330	May 28, 1946	86	1,000	1.41	19.11	693	13.24
1947	1081	8,100	Apr. 6, 1947	78	857	1.21	16.36	990	18.91
1948	1111	13,700	Mar. 17, 1948	32	1,106	1.56	21.19	1,071	20.49
1949	1141	21,000	Dec. 31, 1948	36	863	1.21	16.45	785	14.98
1950	1171	7,690	Mar. 9, 1950	46	861	1.21	16.43	-	-

Not previously published.

123. Wallkill River at New Paltz, N. Y.

Location.--Lat 41°44'50", long 74°05'25", on downstream side of highway bridge in New Paltz, Ulster County.

Drainage area.--739 sq mi (revised).

Gage.--Chain gage. Altitude of gage is 180 ft (from topographic map).

Monthly and yearly mean discharge, in cubic feet per second

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1901	-	-	-	-	-	-	-	-	-	-	2,043	981	-
1902	528	517	3,323	2,139	739	7,188	1,878	913	394	918	742	820	1,691
1903	2,076	704	2,611	1,729	2,910	†3,558	2,340	312	#3,192	1,380	#2,154	†994	#1,992
1904	4,286	-	-	-	-	-	-	-	-	-	-	-	-

† Corrected.
# Not previously published; partly estimated on basis of corrected gage readings.

Monthly and yearly runoff, in inches

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1901	-	-	-	-	-	-	-	-	-	-	3.19	1.48	-
1902	0.82	0.78	5.18	3.34	1.04	11.21	2.84	1.42	0.59	1.43	1.16	1.24	31.05
1903	3.24	1.06	4.07	2.70	4.10	†5.55	3.53	.49	#4.82	2.16	#3.36	†1.50	#36.58
1904	6.69	-	-	-	-	-	-	-	-	-	-	-	-

† Corrected.
# Not previously published; partly estimated on basis of corrected gage readings.

Yearly discharge, in cubic feet per second

Year	W.S.P. no.	Water year ending Sept. 30						Calendar year	
		Momentary maximum		Minimum day	Mean	Per square mile	Runoff in inches	Mean	Runoff in inches
		Discharge	Date						
1902	97	-	-	70	1,691	2.29	31.05	1,760	32.64
1903	97	-	-	121	#1,992	#2.70	#36.58	-	-

Not previously published.

Kill at Pine Bush, N. Y.

74°17'40", on left bank 50 ft downstream from ... of Pine Bush, Orange County, 21 miles down- ... above mouth at Ganahgote.

...ptember 1932, June 1957 to September 1960.

... gage is 305 ft, revised (from topographic map). ... site and datum.

...(57-60), 158 cfs.

...charge, 7,350 cfs Sept. 1, 1927 (gage height, ...), from rating curve extended above 2,300 cfs ... height 8.07 ft, an estimated discharge in ... indirect measurements on Shawangunk Kill at ... discharge in 1955 based on a floodmark at ... Ganahgote; minimum, 2.3 cfs July 21, 22, 1957; ...

... reached a stage of about 12.5 ft, from flood-... runoff at Ganahgote for each flood). Flood ... 11.0 ft, from floodmarks (discharge, 7,200 ... at Ganahgote).

... dams above station.

...scharge, in cubic feet per second

Apr.	May	June	July	Aug.	Sept.	The year
-	-	19.9	10.6	8.04	12.5	-
441	253	41.4	35.2	24.0	32.5	174
371	70.0	56.1	26.3	27.7	23.6	114
354	102	112	66.6	169	295	215

...ly runoff, in inches

Apr.	May	June	July	Aug.	Sept.	The year
-		0.22	0.12	0.09	0.14	-
4.33		.45	.40	.27	.36	23.22
3.31	.9	.61	.30	.31	.26	15.19
4.22	1.15	1.23	.75	1.91	3.12	28.74

...n cubic feet per second

...ng Sept. 30			Calendar year	
Mean	Per square mile	Runoff in inches	Mean	Runoff in inches
-	-	-	-	-
174	1.71	23.22	176	23.47
114	1.12	15.19	139	18.50
215	2.11	28.74	-	-

3715. Wallkill River at Gardiner, N. Y.

Location.--Lat 41°41'10", long 74°09'55", on left bank 400 ft upstream from highway bridge, 500 ft downstream from Shawangunk Kill, and three-quarters of a mile northwest of Gardiner, Ulster County.

Drainage.--711 sq mi.

Records available.--September 1924 to September 1960.

Gage.--Water-stage recorder. Datum of gage is 185.70 ft above mean sea level, adjustment of 1912.

Average discharge.--36 years (1924-60), 1,074 cfs.

Extremes.--1924-60: Maximum discharge, 30,800 cfs Oct. 16, 1955 (gage height, 19.81 ft); minimum, 19 cfs Aug. 9, 1955; minimum gage height, 1.83 ft Aug. 26, 1957.

Remarks.--Large diurnal fluctuations during low and medium flow caused by hydroelectric plant above station. Records of chemical analyses and water temperatures for the period October 1957 to September 1958 are published in report of the Geological Survey.

Monthly and yearly mean discharge, in cubic feet per second

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1951	167	935	1,945	2,047	3,027	2,323	2,426	474	431	385	605	194	1,235
1952	361	2,027	1,776	2,673	1,675	2,689	3,780	2,101	2,516	617	501	1,362	1,834
1953	231	1,349	1,895	2,519	1,622	2,736	2,733	1,546	321	132	111	76.9	1,272
1954	113	365	1,841	622	1,160	1,554	973	1,869	228	85.4	63.0	260	761
1955	132	1,571	1,488	867	1,138	2,244	948	371	232	81.2	3,333	377	1,068
1956	4,217	2,595	493	740	1,955	1,886	3,332	1,601	463	627	184	331	1,531
1957	232	548	1,691	734	1,231	1,271	2,242	455	147	95.9	47.0	72.5	726
1958	122	226	1,724	1,809	999	3,565	3,163	1,925	324	222	136	211	1,212
1959	960	1,048	726	841	1,087	1,859	2,014	569	412	452	289	160	867
1960	1,133	1,295	1,780	1,585	2,017	1,078	2,793	529	500	389	1,620	2,447	1,424

Monthly and yearly runoff, in inches

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1951	0.27	1.47	3.15	3.32	4.43	3.77	3.81	0.77	0.68	0.62	0.98	0.30	23.57
1952	.56	3.16	2.88	4.33	2.54	4.36	5.93	3.41	3.95	1.00	.81	2.14	35.11
1953	.37	2.12	3.07	4.08	2.38	4.44	4.29	2.51	.50	.21	.18	.12	24.27
1954	.16	.57	2.99	1.01	1.70	2.52	1.53	3.03	.36	.14	.10	.41	14.54
1955	.21	2.47	2.41	1.41	1.67	3.64	1.49	.60	.36	.13	5.41	.59	20.39
1956	6.84	4.06	.80	1.20	2.97	3.06	5.23	2.60	.73	1.02	.30	.52	29.33
1957	.38	.86	2.74	1.19	1.80	2.06	3.52	.74	.23	.16	.08	.11	13.67
1958	.20	.35	2.80	3.03	1.46	5.78	4.96	3.12	.51	.36	.22	.33	23.12
1959	1.59	1.64	1.18	1.36	1.59	3.01	3.16	.92	.65	.73	.47	.25	16.55
1960	1.84	2.03	2.89	2.57	3.06	1.75	4.38	.86	.78	.63	2.63	3.84	27.26

Yearly discharge, in cubic feet per second

Year	WSP	Water year ending Sept. 30							Calendar year	
		Momentary maximum		Minimum day	Mean	Per square mile	Runoff in inches		Mean	Runoff in inches
		Discharge	Date							
1950	-	-	-	-	-	-	-		1,056	20.13
1951	1202	12,600	Mar. 31, 1951	65	1,235	1.74	23.57		1,326	25.32
1952	1232	21,200	June 1, 1952	106	1,834	2.58	35.11		1,778	34.03
1953	1272	13,000	Jan. 24, 1953	34	1,272	1.79	24.27		1,176	22.45
1954	1332	8,810	Dec. 7, 1953	32	761	1.07	14.54		832	15.89
1955	1382	30,600	Aug. 19, 1955	21	1,068	1.50	20.39		1,414	27.00
1956	1432	30,800	Oct. 16, 1955	125	1,531	2.15	29.33		1,128	21.61
1957	1502	7,160	Apr. 6, 1957	22	726	1.02	13.67		693	13.24
1958	1552	9,270	Dec. 21, 1957	38	1,212	1.70	23.12		1,207	24.18
1959	1622	7,500	Jan. 22, 1959	65	867	1.22	16.55		990	18.90
1960	1702	10,200	Sept.13, 1959	130	1,424	2.00	27.26		-	-

CENTRAL HUDSON GAS
AND
ELECTRIC CORPORATION
DASHVILLE HYDRO GENERATING PLANT

REPORT
ON
PROPOSED PLANT
RETIREMENT CONSIDERATIONS

MAIN
Chas. T. Main of New York, Inc.

March, 1973

1050-31

HYDROLOGY & FLOOD STUDY

The Dashville Hydro Generating Plant, located on the Wallkill River, commands an area of about 789 square miles and has been subject to many flood situations.

The closest U.S. stream gaging station is at Gardiner and commands an area of 711 square miles and has a period of record extending from 1924. A return period analysis of the annual peak flows was made and the results of this analysis are presented in Table I.

TABLE I
FLOOD FREQUENCY

Return Period - Years	Annual Peaks - cfs
2	9,100
5	14,300
10	18,000
20	21,800
50	27,000
100	31,300
500	41,900
1000	47,200

The flood of September 1960 was analyzed and adopted for use in the derivation of a unit hydrograph for the Wallkill River at the Gardiner gaging site. To this unit hydrograph were applied the appropriate 12-hour rainfall excess values for the Probable Maximum Precipitation (PMP), as taken from the Joint U.S. Weather Bureau - Corps of Engineers Hydrometeorological Report No. 33, and corrected for infiltration losses to obtain a Probable Maximum Flood (PMF) at Gardiner.

A comparison of several major flood peaks showed a significant reduction in these peaks from the Gardiner gage to the Dashville Project. This observation led to the conclusion that valley storage, a very common phenomenon in northerly or northeasterly flowing streams in this area, was the cause for this reduction and so it was investigated further. Profiles of the 1955 storms showed that this storage probably occurred upstream of Perrines' Bridge and

the U.S.C. & G.S. maps confirmed this hypothesis. The hydrograph comparison between Gardiner and the project for the two floods of 1955 served to establish the intervening valley storage quantitiatively so that a storage curve was derived.

It is current practice to reduce the Probable Maximum Flood by 40 percent to 50 percent to give a Standard Project Flood (SPF) for use in assessing spillway adequacy.

Therefore, the PMF discharge ordinates were reduced 40 percent to obtain the SPF inflow hydrographs which were in turn increased by a factor of 1.1 to compensate for the increase in drainage area between the Gardiner gage and assumed point of valley storage. This SPF hydrograph was routed through the valley storage and over the Dashville Project spillway.

The peak SPF inflow into the natural storage was 62,000 cfs which was reduced by this storage to a peak outflow of 38,000 cfs. This outflow hydrograph was then routed over the project spillway with a peak discharge of 38,000 cfs and a maximum headwater surface elevation of 179.0 and a maximum tailwater elevation of 142.0. Predicated upon the foregoing data, the SPF has a theoretical return period as an annual flood of about 500 years.

APPENDIX E

STRUCTURAL STABILITY ANALYSES

00-15-1 (3/78)
Formerly GA-17

NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
DASHVILLE DAM	1		
SUBJECT		**COMPUTED BY**	**DATE**
STRUCTURAL STABILITY ANALYSIS		RLW	4/30/79

MAXIMUM SECTION OF THE DAM

REVISION OF STABILITY PROGRAM RESULTS

SINCE INTERFACE OF ROCK & DAM IS NOT SMOOTH THEN CONCRETE WOULD HAVE TO SHEAR FOR A SLIDING FAILURE TO OCCUR.

$\therefore$ USE THE FOLLOWING FORMULA

$$F.S._{SLIDING} = \frac{\text{RESISTING FORCE} + (S_A)(A)}{\text{DRIVING FORCE}}$$

S_A = SHEARING STRENGTH OF CONCRETE

A = AREA OF BASE

NORMAL CONDITIONS - USING RESULTS FROM STABILTY PROGRAM

$$F.S._{SLIDING} = \frac{66.69 + (.15)\left[\frac{(42\,FT)(1\,FT)(144\,IN^2)}{1\,FT^2}\right]}{61.33} = 15.88$$

ICE LOADING

$$F.S._{SLIDING} = \frac{66.69 + .15\left[\frac{42(1)(144)}{1}\right]}{66.33} = 14.68$$

½ PMF - (WATER LEVEL 13.8' ABOVE CREST)

$$F.S._{SLIDING} = \frac{62.29 + .15\left[\frac{42(1)(144)}{1}\right]}{94.06} = 10.31$$

PMF - (WATER LEVEL 22.8' ABOVE CREST)

$$F.S._{SLIDING} = \frac{62.29 + .15\left[\frac{42(1)(144)}{1}\right]}{115.40} = 8.40$$

00-15-1 (3/78)
Formerly GA-17

NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
DASHVILLE DAM	2		
SUBJECT: STRUCTURAL STABILITY ANALYSIS		COMPUTED BY: RLW	DATE: 5/28/79

MINIMUM SECTION OF THE DAM

REVISION OF STABILITY PROGRAM RESULTS TO ACCOUNT FOR ROUGH SURFACE.

NORMAL CONDITIONS

$$F.S._{SLIDING} = \frac{2.11 + .15\left[\frac{[illegible]}{1}\right]}{1.06} = 266.9$$

ICE LOADING

$$F.S._{SLIDING} = \frac{2.11 + .15\left[\frac{[illegible]}{1}\right]}{6.06} = 46.68$$

½ PMF

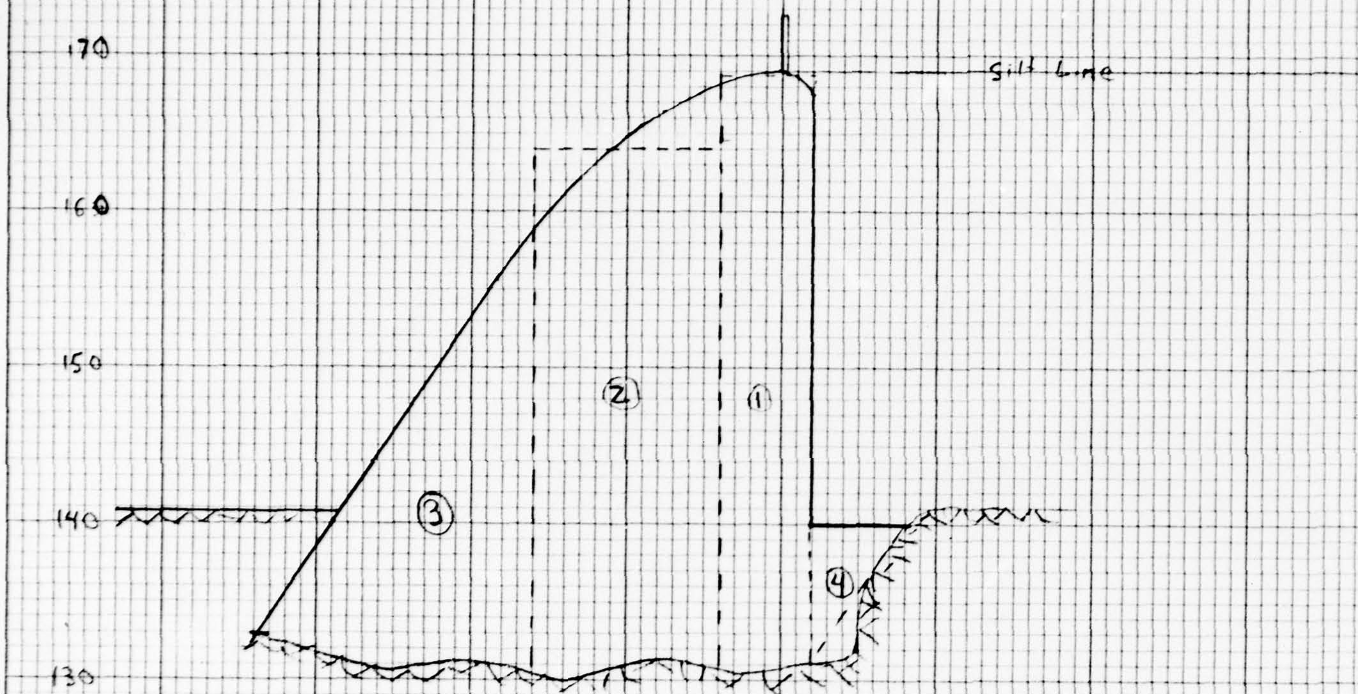
$$F.S._{SLIDING} = \frac{2.11 + .15\left[\frac{[illegible]}{1}\right]}{5.36} = 52.78$$

PMF

$$F.S._{SLIDING} = \frac{2.11 + .15[13(144)]}{8.17} = 34.62$$

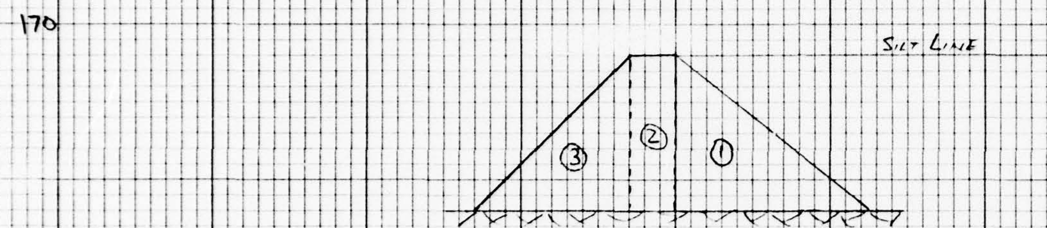
K+E 10 X 10 TO THE INCH • 7 X 10 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A. 46 0700

DASHVILLE DAM N.Y. 76 - MAXIMUM SECTION



SECTION NO.	AREA (ft²)	DISTANCE FROM TOE (ft)
①	(38)(6)=228	18+12+3 = 33
②	(33.5)(12)=402	18+6 = 24
③	½(27)(18)=243	12
④	½(6)(9)=27	18+12+6+2=38

DASHVILLE DAM - N.Y. 76 - MINIMUM SECTION



SEGMENT No.	AREA (ft^2)	DISTANCE FROM TOE (ft)
1	$\frac{1}{2}(5)(7)=17.5$	8.83
2	$1.5(5)=7.5$	5.75
3	$\frac{1}{2}(5)(5)=12.5$	3.33

INPUT TO STABILITY ANALYSIS PROGRAM

INPUT ENTRY	PROGRAM No.
Unit Weight of Dam (K/ft^3)	0
Area of Segment No. 1 (ft^2)	1
Distance from Center of Gravity of Segment No. 1 to Downstream Toe (ft)	2
Area of Segment No. 2 (ft^2)	3
Distance from Center of Gravity of Segment No. 2 to Downstream Toe (ft)	4
Area of Segment No. 3 (ft^2)	5
Distance from Center of Gravity of Segment No. 3 to Downstream Tow (ft)	6
Base Width of Dam (Total) (ft)	7
Height of Dam (ft)	8
Ice Loading (K/L ft.)	9
Coefficient of Sliding	10
Unit Weight of Soil (K/ft^3)	11
Active Soil Coefficient - Ka	12
Passive Soil Coefficient - Kp	13
Height of Water over Top of Dam or Spillway (ft)	14
Height of Soil for Active Pressure (ft)	15
Height of Soil for Passive Pressure (ft)	16
Height of Water in Tailrace Channel (ft)	17
Weight of Water (K/ft^3)	18
Area of Segment No. 4 (ft^2)	19
Distance from Center of Gravity of Segment No. 4 to Downstream Toe (ft)	20
Height of Ice Load or Active Water (ft)	46

Maximum Section

Normal Conditions		Ice Load 5 KSF	
0.15	RCL 1	0.15	RCL 1
228.		228.	
228.	RCL 2	228.	RCL 2
33.		33.	
33.	RCL 3	33.	RCL 3
402.		402.	
402.	RCL 4	402.	RCL 4
24.		24.	
24.	RCL 5	24.	RCL 5
243.		243.	
243.	RCL 6	243.	RCL 6
12.		12.	
12.	RCL 7	12.	RCL 7
42.		42.	
42.	RCL 8	42.	RCL 8
38.		38.	
38.	RCL 9	38.	RCL 9
0.		5.	
0.	RCL 10	5.	RCL 10
0.6		0.6	
0.6	RCL 11	0.6	RCL 11
0.055		0.055	
0.055	RCL 12	0.055	(12
0.41		0.41	
0.41	RCL 13	0.41	RCL 13
5.8		5.8	
5.8	RCL 14	5.8	RCL 14
0.		0.	
0.	RCL 15	0.	RCL 15
38.		38.	
38.	RCL 16	38.	RCL 16
10.		10.	
10.	RCL 17	10.	RCL 17
0.5		0.5	
0.5	RCL 18	0.5	RCL 18
0.0624		0.0624	
0.0624	RCL 19	0.0624	RCL 19
27.		27.	
27.	RCL 20	27.	RCL 20
38.		38.	
38.	RCL 46	38.	RCL 46
38.		38.	

F. of S.

Normal Conditions		Ice Load
1.476960528	OVERTURNING	1.358571127
12.29960563		10.05248399
1.087281128	SLIDING	1.005331512

MAXIMUM SECTION

½ PMF

0.15	RCL 1
228.	
228.	RCL 2
33.	
33.	RCL 3
402.	
402.	RCL 4
24.	
24.	RCL 5
243.	
243.	RCL 6
12.	
12.	RCL 7
42.	
42.	RCL 8
38.	
38.	RCL 9
0.	
0.	RCL 10
0.6	
0.6	RCL 11
0.055	
0.055	RCL 12
0.41	
0.41	RCL 13
5.8	
5.8	RCL 14
13.8	
13.8	RCL 15
38.	
38.	RCL 16
10.	
10.	RCL 17
9.	
9.	RCL 18
0.0624	
0.0624	RCL 19
27.	
27.	RCL 20
38.	
38.	RCL 46
38.	

PMF

0.15	RCL 1
228.	
228.	RCL 2
33.	
33.	RCL 3
402.	
402.	RCL 4
24.	
24.	RCL 5
243.	
243.	RCL 6
12.	
12.	RCL 7
42.	CLR
0.	RCL 8
38.	
38.	RCL 9
0.	
0.	RCL 10
0.6	
0.6	RCL 11
0.055	
0.055	RCL 12
0.41	
0.41	RCL 13
5.8	
5.8	RCL 14
22.8	
22.8	RCL 15
38.	
38.	RCL 16
10.	
10.	RCL 17
15.	
15.	RCL 18
0.0624	
0.0624	RCL 19
27.	
27.	RCL 20
38.	
38.	RCL 46
38.	

F. of S.

½ PMF		PMF
1.0[illegible]045575	OVERTURNING	.93[illegible]873527
2.8[illegible]7296549	SLIDING	-3.3[illegible]555824
~~.[illegible]~~		~~0.[illegible]~~

MINIMUM SECTION

NORMAL CONDITIONS		ICE LOAD 5 KSF	
0.15	RCL 1	0.15	RCL 1
17.5 17.5	RCL 2	17.5 17.5	RCL 2
8.8 8.8	RCL 3	8.8 8.8	RCL 3
7.5 7.5	RCL 4	7.5 7.5	RCL 4
5.7 5.7	RCL 5	5.7 5.7	RCL 5
12.5 12.5	RCL 6	12.5 12.5	RCL 6
3.3 3.3	RCL 7	3.3 3.3	RCL 7
13.5 13.5	RCL 8	13.5 13.5	RCL 8
5. 5.	RCL 9	5. 5.	RCL 9
0. 0.	RCL 10	5. 5.	RCL 10
0.6 0.6	RCL 11	0.6 0.6	RCL 11
0.055 0.055	RCL 12	0.055 0.055	RCL 12
0.41 0.41	RCL 13	0.41 0.41	RCL 13
5.8 5.8	RCL 14	5.8 5.8	RCL 14
0. 0.	RCL 15	0. 0.	RCL 15
5. 5.	RCL 16	5. 5.	RCL 16
0. 0.	RCL 17	0. 0.	RCL 17
0. 0.	RCL 18	0. 0.	RCL 18
0.0624 0.0624	RCL 19	0.0624 0.0624	RCL 19
0. 0.	RCL 20	0. 0.	RCL 20
0. 0.	RCL 46	0. 0.	RCL 46
5.		5.	

F. OF S.

1.722057734	OVERTURNING	.7807751435
4.255010678		-2.848477314
1.988369629	SLIDING	0.348000073

MINIMUM SECTION

½ PMF

0.15 RCL 1
17.5
17.5 RCL 2
8.8
8.8 RCL 3
7.5
7.5 RCL 4
5.7
5.7 RCL 5
12.5
12.5 RCL 6
3.3
3.3 RCL 7
13.5
13.5 RCL 8
5.
5. RCL 9
0.
0. RCL 10
0.6
0.6 RCL 11
0.055
0.055 RCL 12
0.41
0.41 RCL 13
5.8
5.8 RCL 14
13.8
13.8 RCL 15
5.
5. RCL 16
0.
0. RCL 17
0.
0. RCL 18
0.0624
0.0624 RCL 19
0.
0. RCL 20
0.
0. RCL 46
5.

PMF

0.15 RCL 1
17.5
17.5 RCL 2
8.8
8.8 RCL 3
7.5
7.5 RCL 4
5.7
5.7 RCL 5
12.5
12.5 RCL 6
3.3
3.3 RCL 7
13.5
13.5 RCL 8
5.
5. RCL 9
0.
0. RCL 10
0.6
0.6 RCL 11
0.055
0.055 RCL 12
0.41
0.41 RCL 13
5.8
5.8 RCL 14
22.8
22.8 RCL 15
5.
5. RCL 16
0.
0. RCL 17
0.
0. RCL 18
0.0624
0.0624 RCL 19
0.
0. RCL 20
0.
0. RCL 46
5.

½ PMF	F. of S.	PMF
1.133772745		.9270851029
1.196990149	OVERTURNING	-.7978947618
.3933693217	SLIDING	.2582602234

CENTRAL HUDSON GAS
AND
ELECTRIC CORPORATION
DASHVILLE HYDRO GENERATING PLANT

REPORT
ON
PROPOSED PLANT
RETIREMENT CONSIDERATIONS

MAIN
Chas. T. Main of New York, Inc.

March, 1973

1050-31

STRUCTURAL STABILITY ANALYSIS

A structural stability analysis of all water retaining structures in their final altered condition was performed to verify that the structures would be stable under the assumed loadings as presented in Table II. The results of the analysis, as presented in Table IV, indicate that the structures are stable under the loadings as assumed. Tables II and III list the loading cases checked and values and assumptions used in the above analysis.

The safety factors of the structures were checked with respect to overturning and sliding at their bases, and computations were made to check the foundation pressures at this elevation.

The safety factor with respect to overturning is the ratio of the forces (the weight of structure) times their lever arms (moments) tending to prevent the structure from tipping to the forces (moments) tending to tip the structure (the water pressure exerted on the upstream face and beneath the structure). Any safety factor that is equal to 1.0 would theoretically indicate that the structure is stable, with any lesser value placing it at the verge of being unstable. By refering to Table IV, under the column headed Σ Mr/Σ Mo, for all cases considered, the structures are stable with respect to overturning.

The safety factor with respect to sliding including shear-friction resistance is the ratio of the forces tending to resist sliding; namely, the frictional resistance due to the net weight of the structure sliding along its base and the resistance due to the shearing strength developed between the structure and its rock foundation, and the forces tending to promote sliding; namely, the water pressure at the upstream face. It is normally accepted that this ratio be as a minimum 5.0. By refering to Table IV, under the column headed $S_{s\text{-}f}$ for all cases considered, the structures are stable with respect to sliding.

During the last field inspection it was noted that silt had accumulated to within 1-1/2 feet from the top of the flashboards at the left abutment and was impeding their removal. It was decided that the additional pressure resulting from the silt should be incorporated into the stability analysis to reflect this observed condition.

TABLE II
CASES USED
STABILITY ANALYSIS

Case I	Normal Levels (proposed) H.W.L. = 170.0 T.W.L. = 133.75 Uplift Included
Case II	Standard Project Flood Water Levels H.W.L. = 179.0 T.W.L. = 142.0 Uplift Included

TABLE III
VALUES AND ASSUMPTIONS
STABILITY ANALYSIS
CONCRETE SECTIONS

1. Nomenclature

ΣH = Summation of Horizontal Forces

ΣV = Summation of Vertical Forces

ΣM_R = Summation of Resisting Moments

ΣM_o = Summation of Overturning Moments

$\dfrac{\Sigma M_R}{\Sigma M_o}$ = Factor of Safety Against Overturning

$\dfrac{\Sigma H}{\Sigma V}$ = Coefficient of Sliding

2. Unit weight of concrete - 150 lbs/cu. ft.

3. Unit weight of water - 62.4 lbs/cu. ft.

4. Silt pressure: The horizontal silt pressure was assumed to be equal to $14(h_s)^2$ lbs, where h_s = height of silt. Unit weight of silt -90 lbs/cu. ft.

5. Uplift Pressure: The pressure was assumed to vary linearly from full headwater pressure at the upstream side to full tailwater pressure at the downstream side taken over 100% of the base area.

6. Sliding (Shear Included) For a discussion and explanation of terms, see Hydroelectric Handbook by Creager and Justin, John Wiley & Sons, Inc., Second Edition - Page 341.

$$S_{s\text{-}f} = \frac{f \Sigma V + r\ Sa\ A}{\Sigma H}$$

Where:

$S_{s\text{-}f}$ = Shear Friction Factor of Safety
f = 0.75; r=0.5; Sa=380 psi; A=Area of base

TABLE IV

STABILITY SUMMARY

SECTION	CONDITION	BASE ELEV.	BASE LENGTH	ΣH (KIPS)	ΣV (KIPS)	$\frac{\Sigma H}{\Sigma V}$	S_{s-f}	RESULTANT FROM DOWNSTREAM	ΣM_R (K-FT)	ΣM_O (K-FT)	$\frac{\Sigma M_R}{\Sigma M_O}$	BASE STRESS (PSI) UPSTREAM	BASE STRESS (PSI) DOWNSTREAM
POWERHOUSE INTAKE STRUCTURE	I - NORMAL	115.0	59.92	2739	7359	0.37	35.7	16.0	364,842	247,030	1.48	- 4.6	38.5
	II - FLOOD	115.0	59.92	3927	7105	0.55	24.8	11.3	444,864	364,705	1.22	-11.3	44.4
OGEE SPILLWAY	I - NORMAL	133.3	39.0	53.8	90.4	0.60	21.1	15.6	3,169	1,756	1.80	9	23
	II - FLOOD	133.3	39.0	71.5	76.7	0.93	15.7	10.3	3,357	2,568	1.31	- 5.7	33

NOTE:

1. NEGATIVE BASE STRESS INDICATES TENSION.
2. THE SILT LOADING FOR THE POWERHOUSE INTAKE STRUCTURE WAS NOT CONSIDERED SINCE THE OMISSION OF THE LOADING WAS FOUND TO BE A MORE CRITICAL CONDITION.

March 1973

CENTRAL HUDSON GAS & ELECTRIC CORP.
POUGHKEEPSIE, NEW YORK
DASHVILLE HYDRO

CONCRETE SECTIONS ANALYZED FOR STABILITY

MAIN
CHAS. T. MAIN OF NEW YORK, INC.

APPENDIX F

REFERENCES

REFERENCES

1) Conservation Commission - New York State; Dashville Dam;
   a) Computation sheets - October 23-25, 1919
   b) Memorandum - November 3, 1919

2) US Department of the Interior, Bureau of Reclamation; Design of Small Dams, 2nd edition (Rev. reprint), 1977

3) US Geological Survey; Compilation of Records of Surface Waters of the United States, Part 1-B North Atlantic Slope Basins;

   Water Supply Paper 1302 (Through September 1950), 1960

   Water Supply Paper 1722 (October 1950 to September 1960),1964

4) George, Thomas S. and Taylor, Robert S.; Lower Hudson River Basin Hydrologic Flood Routing Model, for the Department of the Army, New York District, Corps of Engineers, Water Resources Engineers Inc, January 1977

5) L.R. Kimball and Associates; Phase 1 Inspection Report - Sturgeon Pool Dam NY-75, for the Department of the Army, New York District, Corps of Engineers, September 1978

6) H.W. King and E.F. Brater; Handbook of Hydraulics, 5th edition, McGraw-Hill, 1963

7) C.T. Main of New York, Inc.; Dashville Hydro Generating Plant-Report on Proposed Plant Retirement Considerations, for Central Hudson Gas and Electric Corporation, March 1973

8) University of the State of New York; Geology of New York, Education Leaflet 20, reprinted 1973

APPENDIX G

CORPS OF ENGINEERS GUIDELINES

Reclamation and Soil Conservation Service. Many other agencies, educational facilities and private consultants can also provide expert advice. Regardless of where such expertise is based, the qualification of those individuals offering to provide it should be carefully examined and evaluated.

4.3.4. Freeboard Allowances. Guidelines on specific minimum freeboard allowances are not considered appropriate because of the many factors involved in such determinations. The investigator will have to assess the critical parameters for each project and develop its minimum requirement. Many projects are reasonably safe without freeboard allowance because they are designed for overtopping, or other factors minimize possible overtopping. Conversely, freeboard allowances of several feet may be necessary to provide a safe condition. Parameters that should be considered include the duration of high water levels in the reservoir during the design flood; the effective wind fetch and reservoir depth available to support wave generation; the probability of high wind speed occurring from a critical direction; the potential wave runup on the dam based on roughness and slope; and the ability of the dam to resist erosion from overtopping waves.

4.4. Stability Investigations. The Phase II stability investigations should be compatible with the guidelines of this paragraph.

4.4.1. Foundation and Material Investigations. The scope of the foundation and materials investigation should be limited to obtaining the information required to analyze the structural stability and to investigate any suspected condition which would adversely affect the safety of the dam. Such investigations may include borings to obtain concrete, embankment, soil foundation, and bedrock samples; testing specimens from these samples to determine the strength and elastic parameters of the materials, including the soft seams, joints, fault gouge and expansive clays or other critical materials in the foundation; determining the character of the bedrock including joints, bedding planes, fractures, faults, voids and caverns, and other geological irregularities; and installing instruments for determining movements, strains, suspected excessive internal seepage pressures, seepage gradients and uplift forces. Special investigations may be necessary where suspect rock types such as limestone, gypsum, salt, basalt, claystone, shales or others are involved in foundations or abutments in order to determine the extent of cavities, piping or other deficiencies in the rock foundation. A concrete core drilling program should be undertaken only when the existence of significant structural cracks is suspected or the general qualitative condition of the concrete is in doubt. The tests of materials will be necessary only where such data are lacking or are outdated.

4.4.2. Stability Assessment. Stability assessments should utilize in situ properties of the structure and its foundation and pertinent geologic

information. Geologic information that should be considered includes groundwater and seepage conditions; lithology, stratigraphy, and geologic details disclosed by borings, "as-built" records, and geologic interpretation; maximum past overburden at site as deduced from geologic evidence; bedding, folding and faulting; joints and joint systems; weathering; slickensides, and field evidence relating to slides, faults, movements and earthquake activity. Foundations may present problems where they contain adversely oriented joints, slickensides or fissured material, faults, seams of soft materials, or weak layers. Such defects and excess pore water pressures may contribute to instability. Special tests may be necessary to determine physical properties of particular materials. The results of stability analyses afford a means of evaluating the structure's existing resistance to failure and also the effects of any proposed modifications. Results of stability analyses should be reviewed for compatibility with performance experience when possible.

4.4.2.1. Seismic Stability. The inertial forces for use in the conventional equivalent static force method of analysis should be obtained by multiplying the weight by the seismic coefficient and should be applied as a horizontal force at the center of gravity of the section or element. The seismic coefficients suggested for use with such analyses are listed in Figures 1 through 4. Seismic stability investigations for all high hazard category dams located in Seismic Zone 4 and high hazard dams of the hydraulic fill type in Zone 3 should include suitable dynamic procedures and analyses. Dynamic analyses for other dams and higher seismic coefficients are appropriate if in the judgment of the investigating engineer they are warranted because of proximity to active faults or other reasons. Seismic stability investigations should utilize "state-of-the-art" procedures involving seismological and geological studies to establish earthquake parameters for use in dynamic stability analyses and, where appropriate, the dynamic testing of materials. Stability analyses may be based upon either time-history or response spectra techniques. The results of dynamic analyses should be assessed on the basis of whether or not the dam would have sufficient residual integrity to retain the reservoir during and after the greatest or most adverse earthquake which might occur near the project location.

4.4.2.2. Clay Shale Foundation. Clay shale is a highly overconsolidated sedimentary rock comprised predominantly of clay minerals, with little or no cementation. Foundations of clay shales require special measures in stability investigations. Clay shales, particularly those containing montmorillonite, may be highly susceptible to expansion and consequent loss of strength upon unloading. The shear strength and the resistance to deformation of clay shales may be quite low and high pore water pressures may develop under increase in load. The presence of slickensides in clay shales is usually an indication of low shear stength. Prediction

AD-A075 891 NEW YORK STATE DEPT OF ENVIRONMENTAL CONSERVATION ALBANY F/G 13/2
NATIONAL DAM SAFETY PROGRAM DASHVILLE DAM, INVENTORY NUMBER (NY--ETC(U)
SEP 79 G KOCH DACW51-79-C-0001

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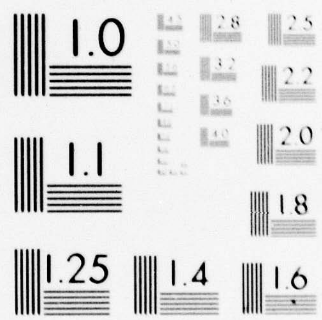


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

of field behavior of clay shales should not be based solely on results of conventional laboratory tests since they may be misleading. The use of peak shear strengths for clay shales in stability analyses may be unconservative because of nonuniform stress distribution and possible progressive failures. Thus the available shear resistance may be less than if the peak shear strength were mobilized simultaneously along the entire failure surface. In such cases, either greater safety factors or residual shear strength should be used.

4.4.3. Embankment Dams.

4.4.3.1. Liquefaction. The phenomenon of liquefaction of loose, saturated sands and silts may occur when such materials are subjected to shear deformation or earthquake shocks. The possibility of liquefaction must presently be evaluated on the basis of empirical knowledge supplemented by special laboratory tests and engineering judgment. The possibility of liquefaction in sands diminishes as the relative density increases above approximately 70 percent. Hydraulic fill dams in Seismic Zones 3 and 4 should receive particular attention since such dams are susceptible to liquefaction under earthquake shocks.

4.4.3.2. Shear Failure. Shear failure is one in which a portion of an embankment or of an embankment and foundation moves by sliding or rotating relative to the remainder of the mass. It is conventionally represented as occurring along a surface and is so assumed in stability analyses, although shearing may occur in a zone of substantial thickness. The circular arc or the sliding wedge method of analyzing stability, as pertinent, should be used. The circular arc method is generally applicable to essentially homogeneous embankments and to soil foundations consisting of thick deposits of fine-grained soil containing no layers significantly weaker than other strata in the foundation. The wedge method is generally applicable to rockfill dams and to earth dams on foundations containing weak layers. Other methods of analysis such as those employing complex shear surfaces may be appropriate depending on the soil and rock in the dam and foundation. Such methods should be in reputable usage in the engineering profession.

4.4.3.3. Loading Conditions. The loading conditions for which the embankment structures should be investigated are (I) Sudden drawdown from spillway crest elevation or top of gates, (II) Partial pool, (III) Steady state seepage from spillway crest elevation or top of gate elevation, and (IV) Earthquake. Cases I and II apply to upstream slopes only; Case III applies to downstream slopes; and Case IV applies to both upstream and downstream slopes. A summary of suggested strengths and safety factors are shown in Table 4.

4.4.3.6. Seepage Analyses. Review and modifications to original seepage design analyses should consider conditions observed in the field inspection and piezometer instrumentation. A seepage analysis should consider the permeability ratios resulting from natural deposition and from compaction placement of materials with appropriate variation between horizontal and vertical permeability. An underseepage analysis of the embankment should provide a critical gradient factor of safety for the maximum head condition of not less than 1.5 in the area downstream of the embankment.

$$F.S = i_c/i = \frac{H_c/D_b}{H/D_b} = D_b \frac{(\gamma_m - \gamma_w)}{H \gamma_w} \qquad (2)$$

i_c = Critical gradient

i = Design gradient

H = Uplift head at downstream toe of dam measured above tailwater

H_c = The critical uplift

D_b = The thickness of the top impervious blanket at the downstream toe of the dam

γ_m = The estimated saturated unit weight of the material in the top impervious blanket

γ_w = The unit weight of water

Where a factor of safety less than 1.5 is obtained the provision of an underseepage control system is indicated. The factor of safety of 1.5 is a recommended minimum and may be adjusted by the responsible engineer based on the competence of the engineering data.

4.4.4. Concrete Dams and Appurtenant Structures.

4.4.4.1. Requirements for Stability. Concrete dams and structures appurtenant to embankment dams should be capable of resisting overturning, sliding and overstressing with adequate factors of safety for normal and maximum loading conditions.

4.4.4.2. Loads. Loadings to be considered in stability analyses include the water load on the upstream face of the dam; the weight of the structure; internal hydrostatic pressures (uplift) within the body of the dam, at the base of the dam and within the foundation; earth and silt loads; ice pressure, seismic and thermal loads, and other loads as applicable. Where tailwater or backwater exists on the downstream side of the structure it should be considered, and assumed uplift pressures should be compatible with drainage provisions and uplift measurements if available. Where applicable, ice pressure should be applied to the contact surface of the structure at normal pool elevation. A unit pressure of not more than 5,000 pounds per square foot should be used. Normally, ice thickness should not be assumed greater than two feet. Earthquake forces should consist of the inertial forces due to the horizontal acceleration of the dam itself and hydrodynamic forces resulting from the reaction of the reservoir water against the structure. Dynamic water pressures for use in conventional methods of analysis may be computed by means of the "Westergaard Formula" using the parabolic approximation (H.M. Westergaard, "Water Pressures on Dams During Earthquakes," Trans., ASCE, Vol 98, 1933, pages 418-433), or similar method.

4.4.4.3. Stresses. The analysis of concrete stresses should be based on in situ properties of the concrete and foundation. Computed maximum compressive stresses for normal operating conditions in the order of 1/3 or less of in situ strengths should be satisfactory. Tensile stresses in unreinforced concrete should be acceptable only in locations where cracks will not adversely affect the overall performance and stability of the structure. Foundation stresses should be such as to provide adequate safety against failure of the foundation material under all loading conditions.

4.4.4.4. Overturning. A gravity structure should be capable of resisting all overturning forces. It can be considered safe against overturning if the resultant of all combinations of horizontal and vertical forces, excluding earthquake forces, acting above any horizontal plane through the structure or at its base is located within the middle third of the section. When earthquake is included the resultant should fall within the limits of the plane or base, and foundation pressures must be acceptable. When these requirements for location of the resultant are not satisfied the investigating engineer should assess the importance to stability of the deviations.

4.4.4.5. Sliding. Sliding of concrete gravity structures and of abutment and foundation rock masses for all types of concrete dams should be evaluated. by the shear-friction resistance concept. The available sliding resistance is compared with the driving force which tends to induce sliding to arrive at a sliding stability safety factor. The investigation should be made along all potential sliding paths. The critical path is that plane or combination of planes which offers the least resistance.

4.4.4.5.1. Sliding Resistance. Sliding resistance is a function of the unit shearing strength at no normal load (cohesion) and the angle of friction on a potential failure surface. It is determined by computing the maximum horizontal driving force which could be resisted along the sliding path under investigation. The following general formula is obtained from the principles of statics and may be derived by resolving forces parallel and perpendicular to the sliding plane:

$$R_R = V \tan(\phi + \alpha) + \frac{cA}{\cos\alpha(1 - \tan\phi \tan\alpha)} \qquad (3)$$

where

R_R = Sliding Resistance (maximum horizontal driving force which can be resisted by the critical path)

ϕ = Angle of internal friction of foundation material or, where applicable, angle of sliding friction

V = Summation of vertical forces (including uplift)

c = Unit shearing strength at zero normal loading along potential failure plane

A = Area of potential failure plane developing unit shear strength "c"

α = Angle between inclined plane and horizontal (positive for uphill sliding)

For sliding downhill the angle α is negative and Equation (1) becomes:

$$R_R = V \tan(\phi - \alpha) + \frac{cA}{\cos\alpha\,(1 + \tan\phi \tan\alpha)} \qquad (4)$$

When the plane of investigation is horizontal, and the angle α is zero and Equation (1) reduced to the following:

$$R_R = V \tan\phi + cA \qquad (5)$$

4.4.4.5.2. Downstream Resistance. When the base of a concrete structure is embedded in rock or the potential failure plane lies below the base, the passive resistance of the downstream layer of rock may sometimes be utilized for sliding resistance. Rock that may be subjected to high velocity water scouring should not be used. The magnitude of the downstream resistance is the lesser of (a) the shearing resistance along the continuation of the potential sliding plane until it daylights or (b) the resistance available from the downstream rock wedge along an inclined plane. The theoretical resistance offered by the passive wedge can be computed by a formula equivalent to formula (3):

$$P_p = W \tan(\phi + \alpha) + \frac{cA}{\cos\alpha\,(1 - \tan\phi \tan\alpha)} \qquad (6)$$

P_p = passive resistance of rock wedge

W = weight (buoyant weight if applicable) of downstream rock wedge above inclined plane of resistance, plus any superimposed loads

ϕ = angle of internal friction or, if applicable, angle of sliding friction

α = angle between inclined failure plane and horizontal

c = unit shearing strength at zero normal load along failure plane

A = area of inclined plane of resistance

When considering cross-bed shear through a relatively shallow, competent rock strut, without adverse jointing or faulting, W and α may be taken at zero and 45^{o}, respectively, and an estimate of passive wedge resistance per unit width obtained by the following equation:

$$P_p = 2\,cD \qquad (7)$$

where

D = Thickness of the rock strut

4.4.4.5.3. Safety Factor. The shear-friction safety factor is obtained by dividing the resistance R_R by H, the summation of horizontal service

loads to be applied to the structure:

$$S_{s-f} = \frac{R_R}{H} \tag{8}$$

When the downstream passive wedge contributes to the sliding resistance, the shear fruction safety factor formula becomes:

$$S_{s-f} = \frac{R_R + P_p}{H} \tag{9}$$

The above direct superimposition of passive wedge resistance is valid only if shearing rigidities of the foundation components are similar. Also, the compressive strength and buckling resistance of the downstream rock layer must be sufficient to develop the wedge resistance. For example, a foundation with closely spaced, near horizontal, relatively weak seams might not contain sufficient buckling strength to develop the magnitude of wedge resistance computed from the cross-bed shear strength. In this case wedge resistance should not be assumed without resorting to special treatment (such as installing foundation anchors). Computed sliding safety factors approximating 3 or more for all loading conditions without earthquake, and 1.5 including earthquake, should indicate satisfactory stability, depending upon the reliability of the strength parameters used in the analyses. In some cases when the results of comprehensive foundation studies are available, smaller safety factors may be acceptable. The selection of shear strength parameters should be fully substantiated. The bases for any assumptions; the results of applicable testing, studies and investigations; and all pre-existing, pertinent data should be reported and evaluated.

APPENDIX G

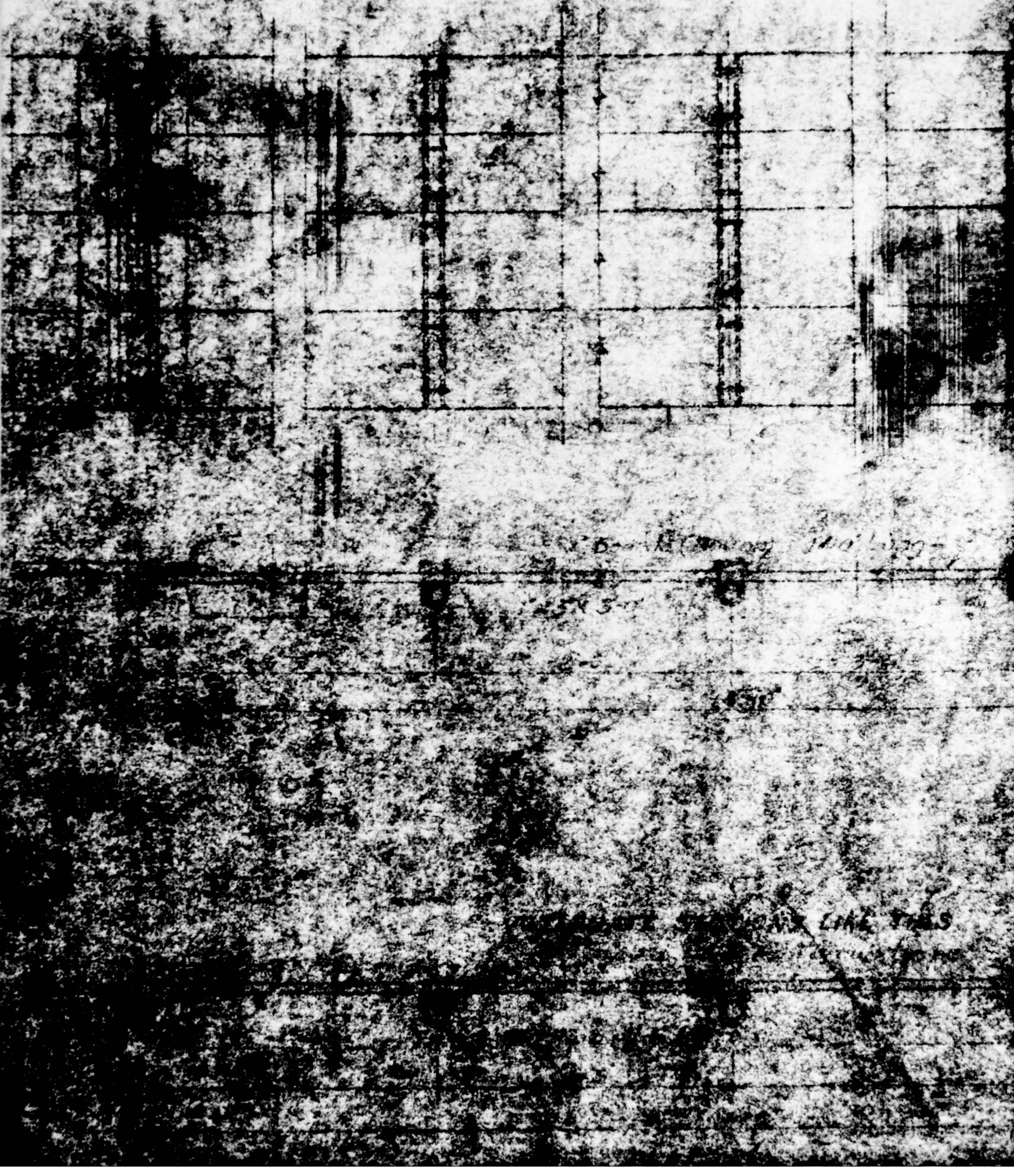
DRAWINGS

DAM SITE

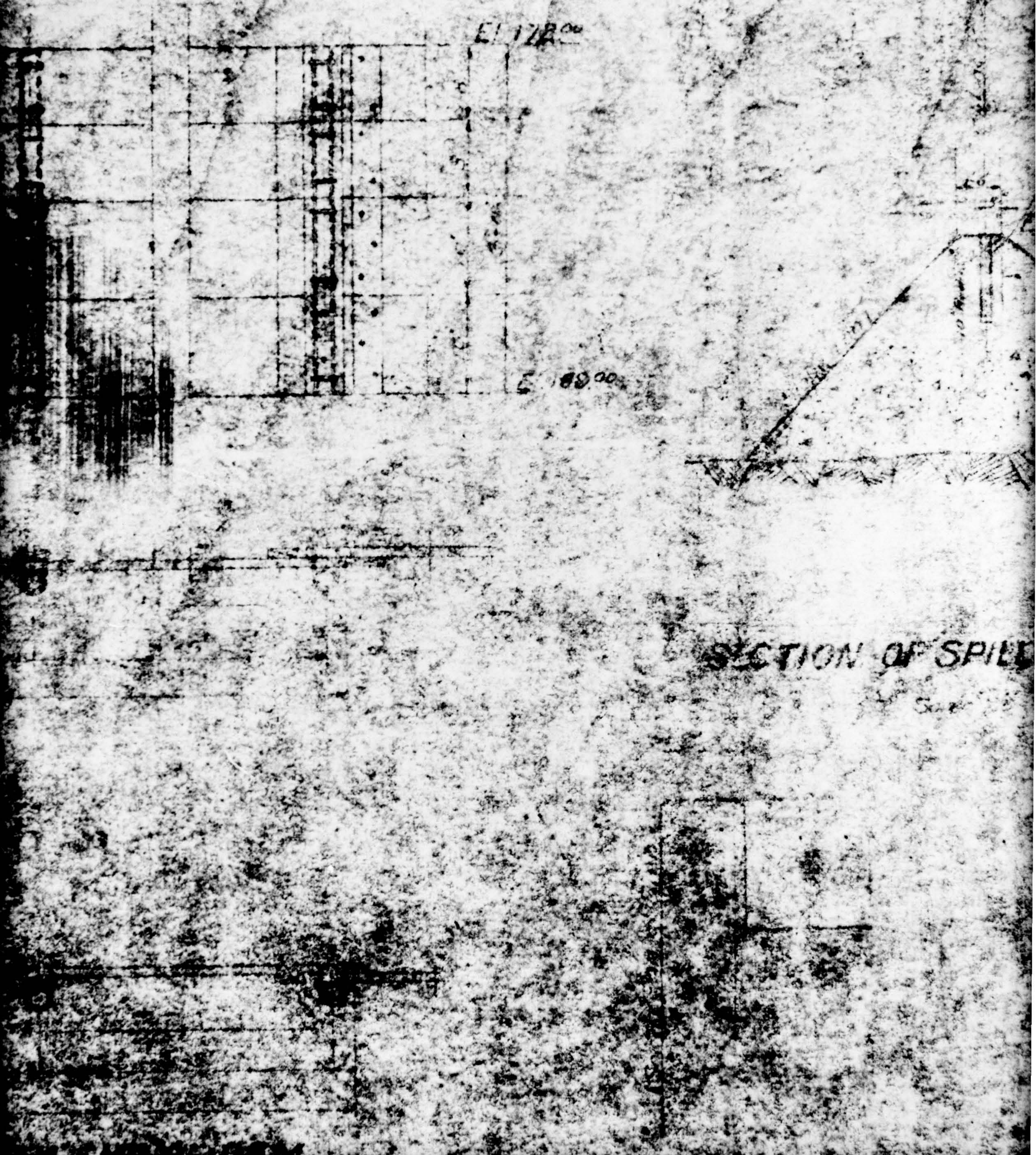
VICINITY MAP

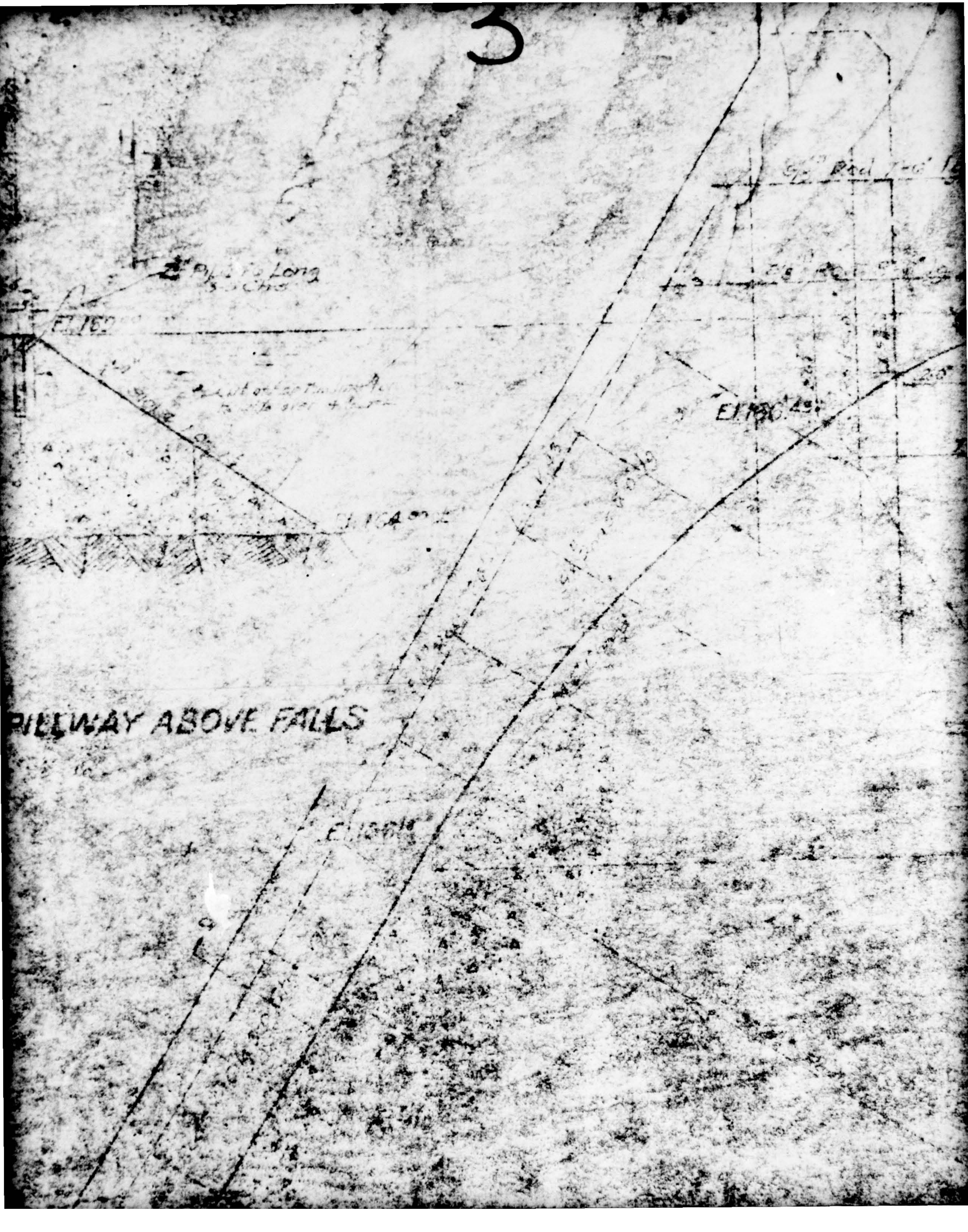
DASHVILLE DAM

1

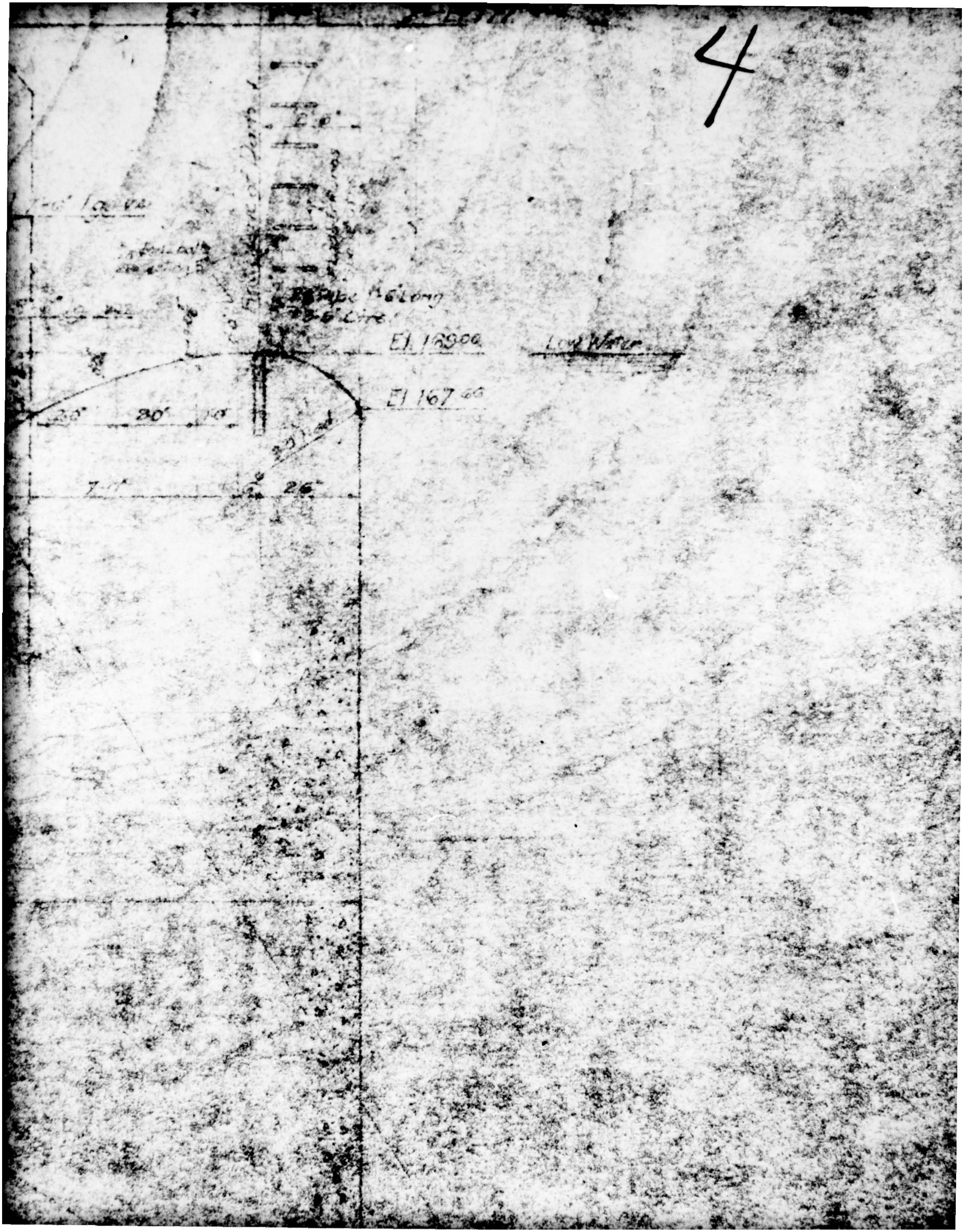


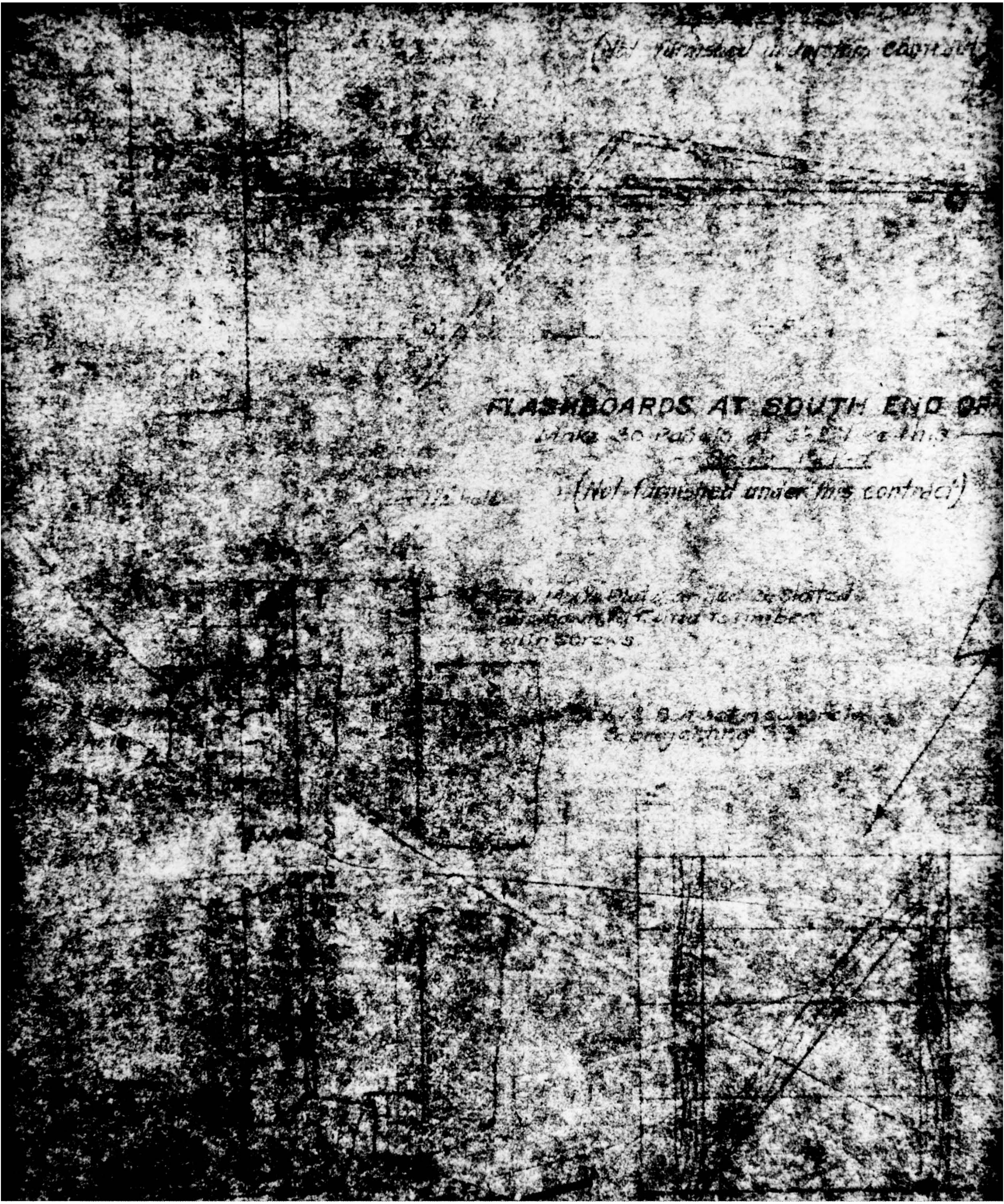
2




3
PILLWAY ABOVE FALLS

4

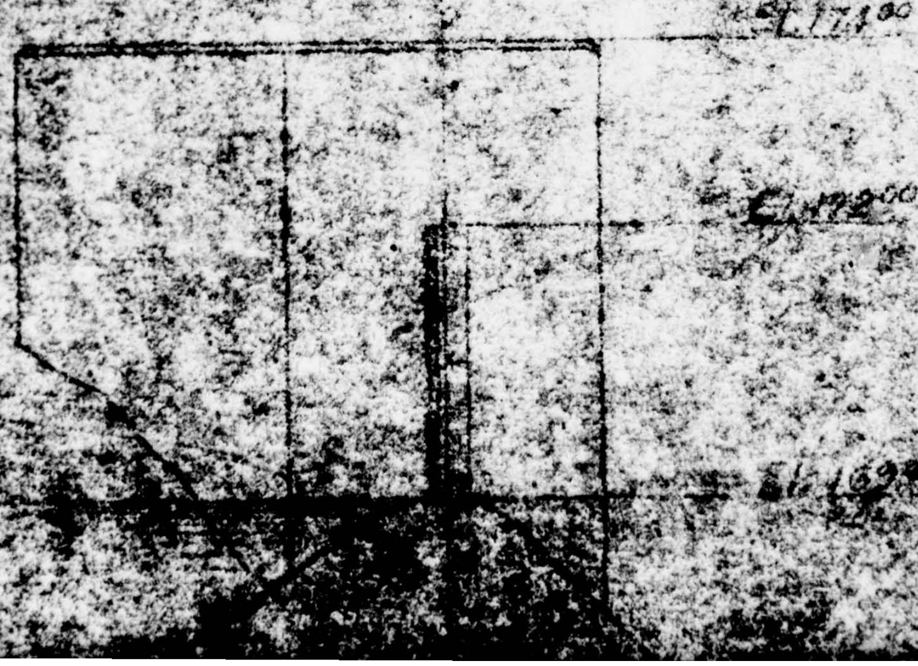


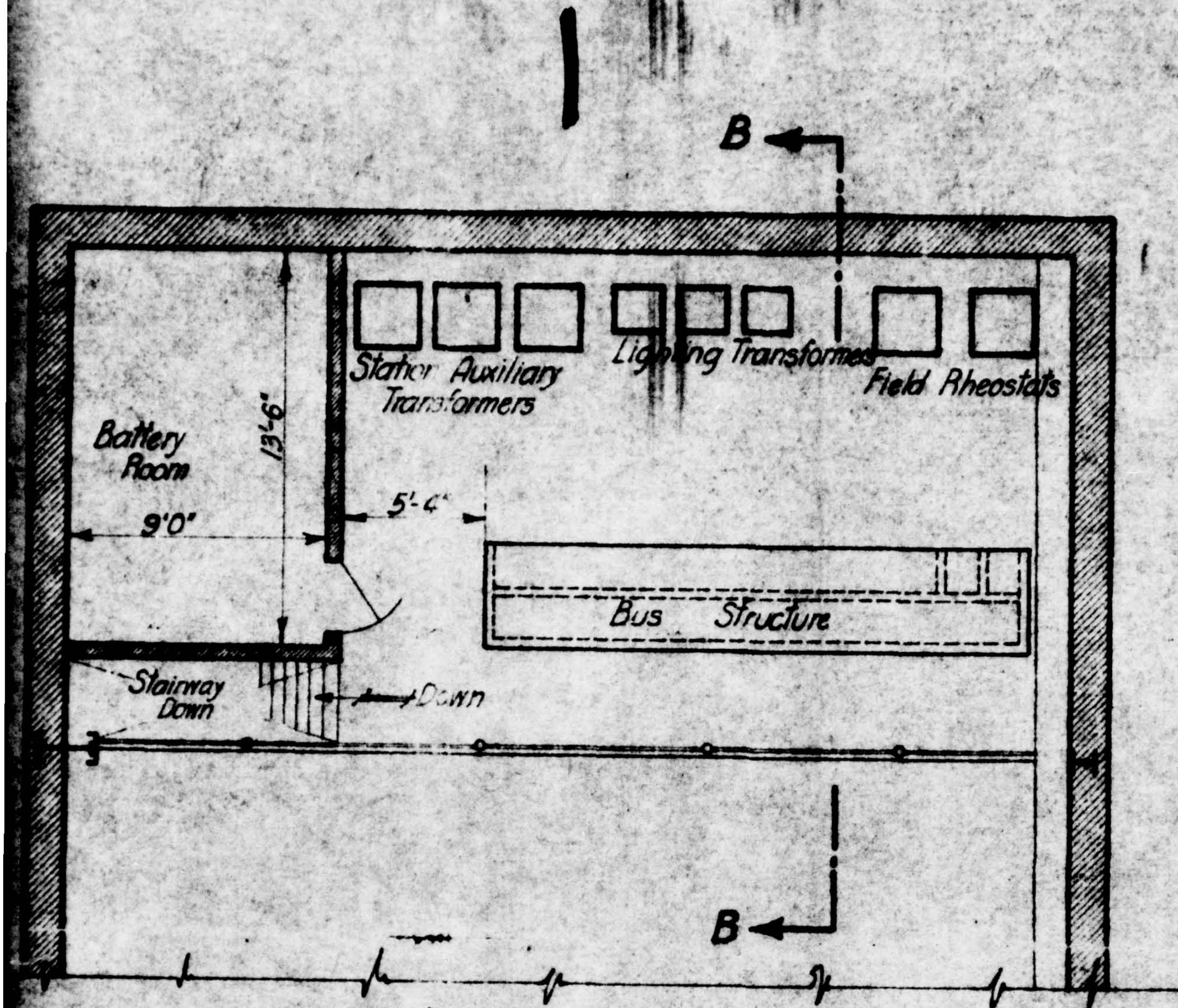

FLASHBOARDS AT SOUTH END OF
(Not furnished under this contract)

6

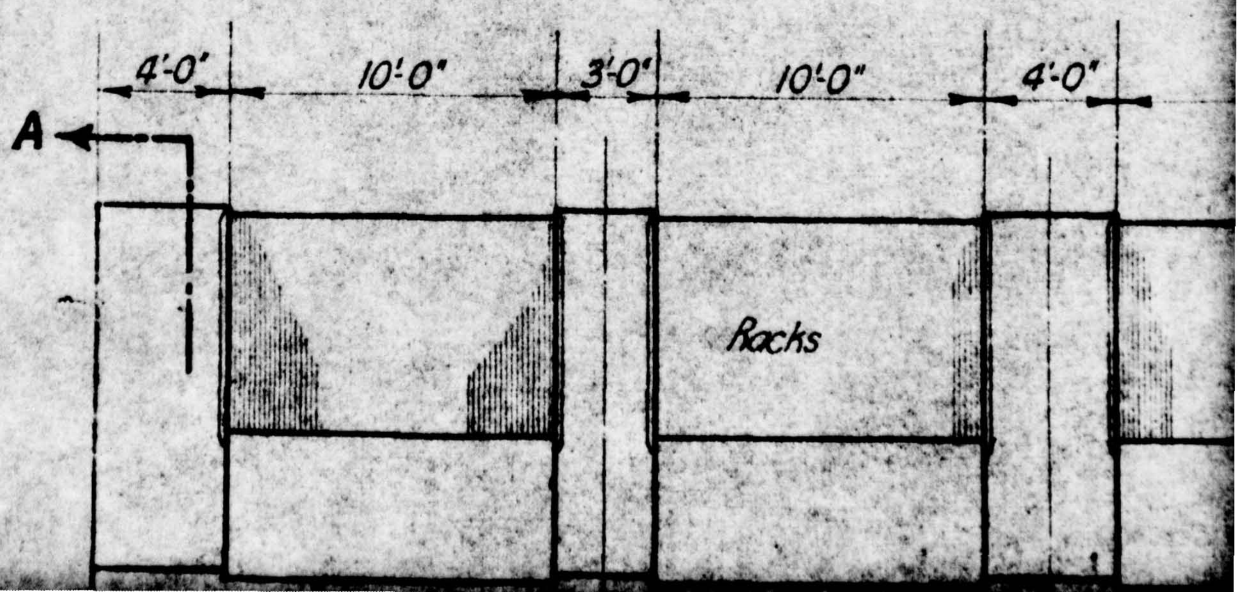
MAXIMUM SECTION OF SPILLWAY

Scale

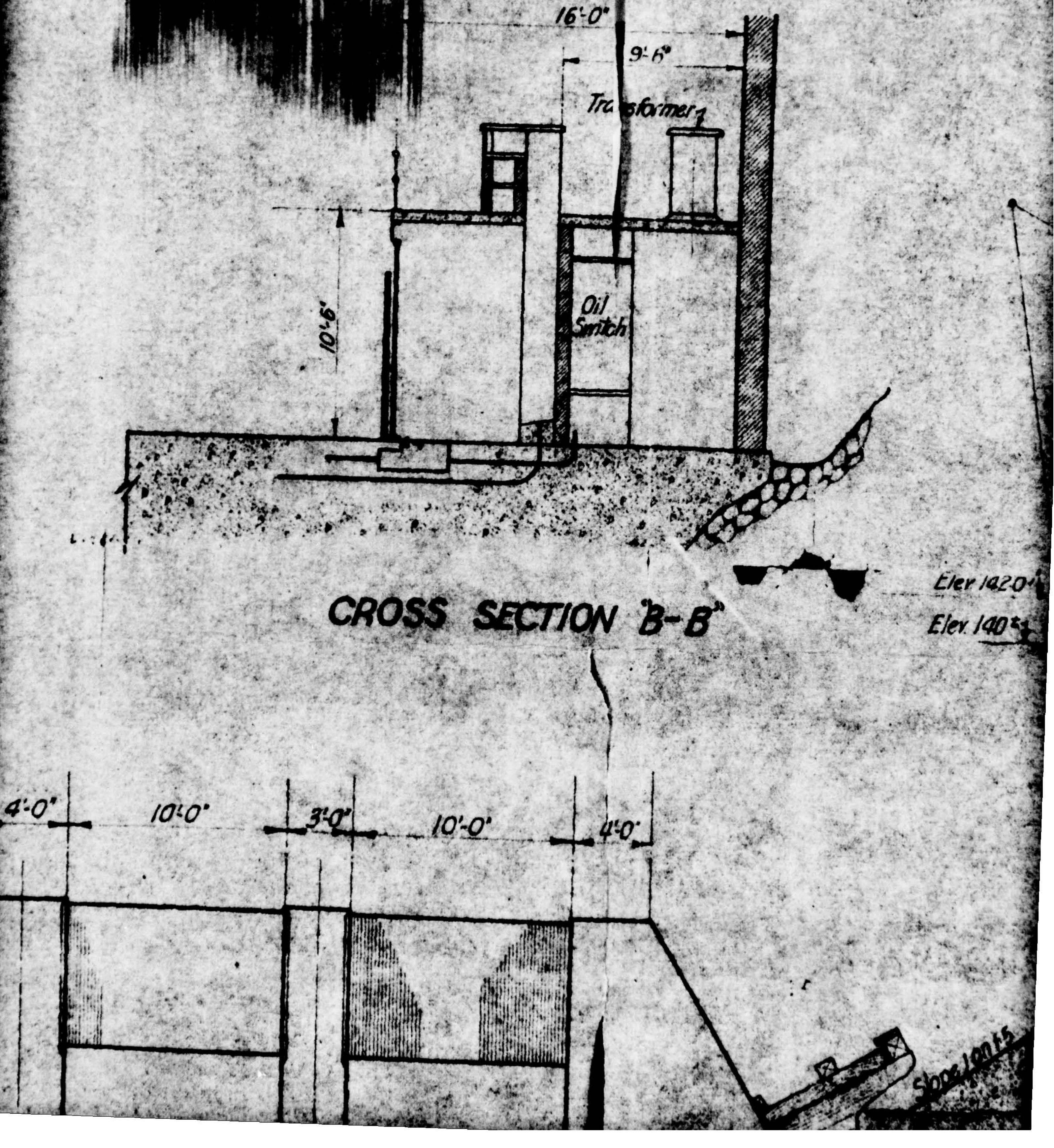




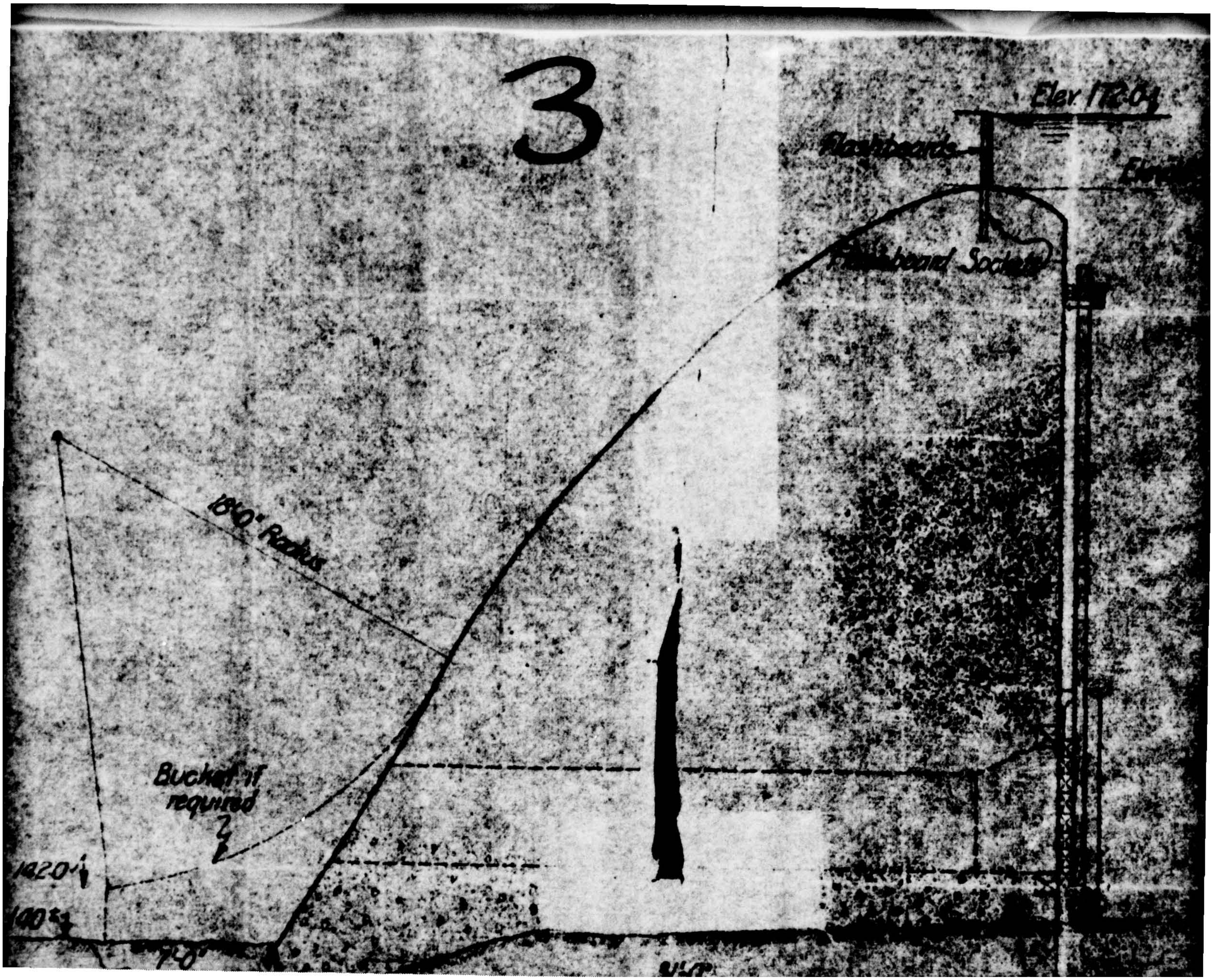
PLAN OF GALLERY



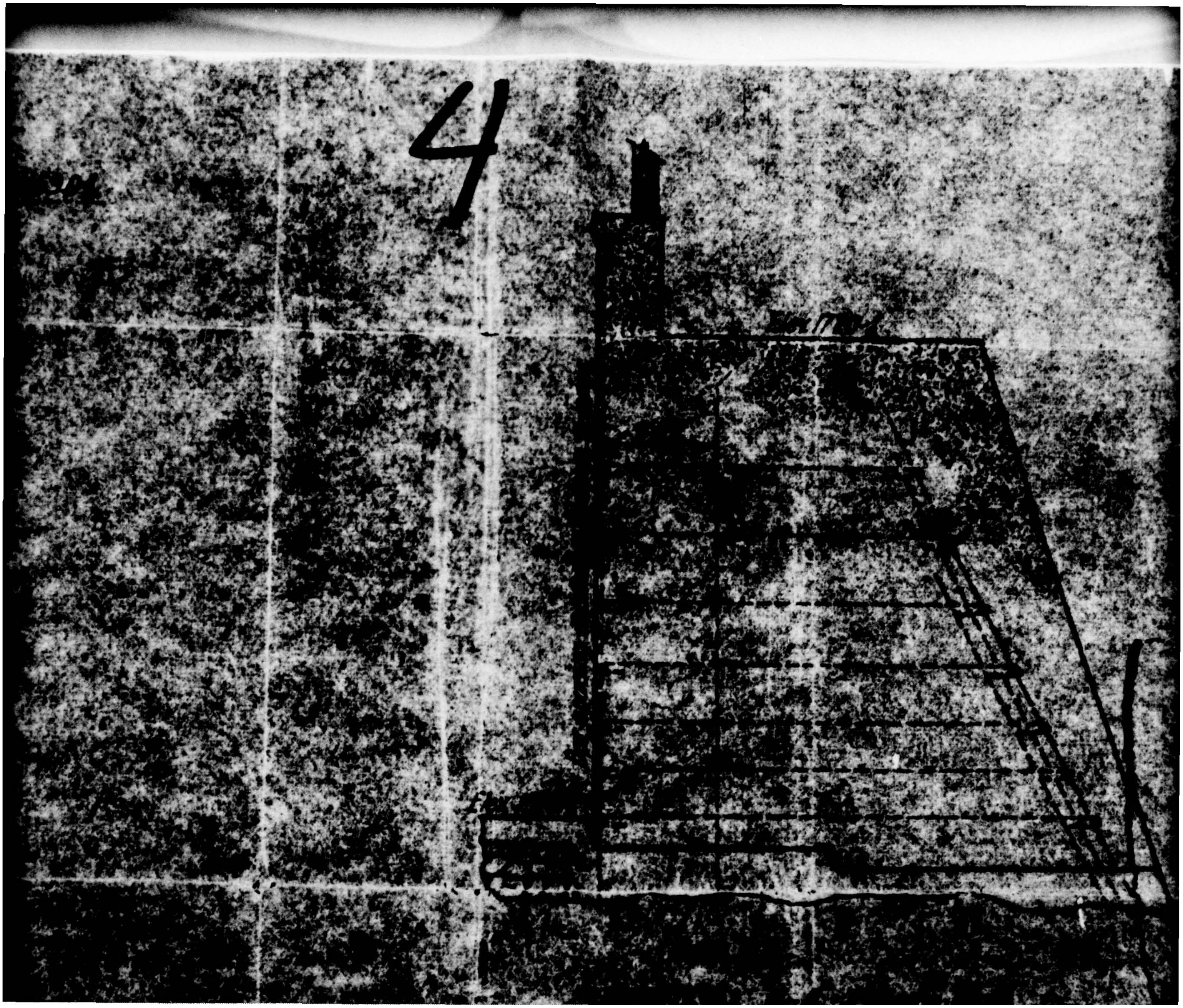
2

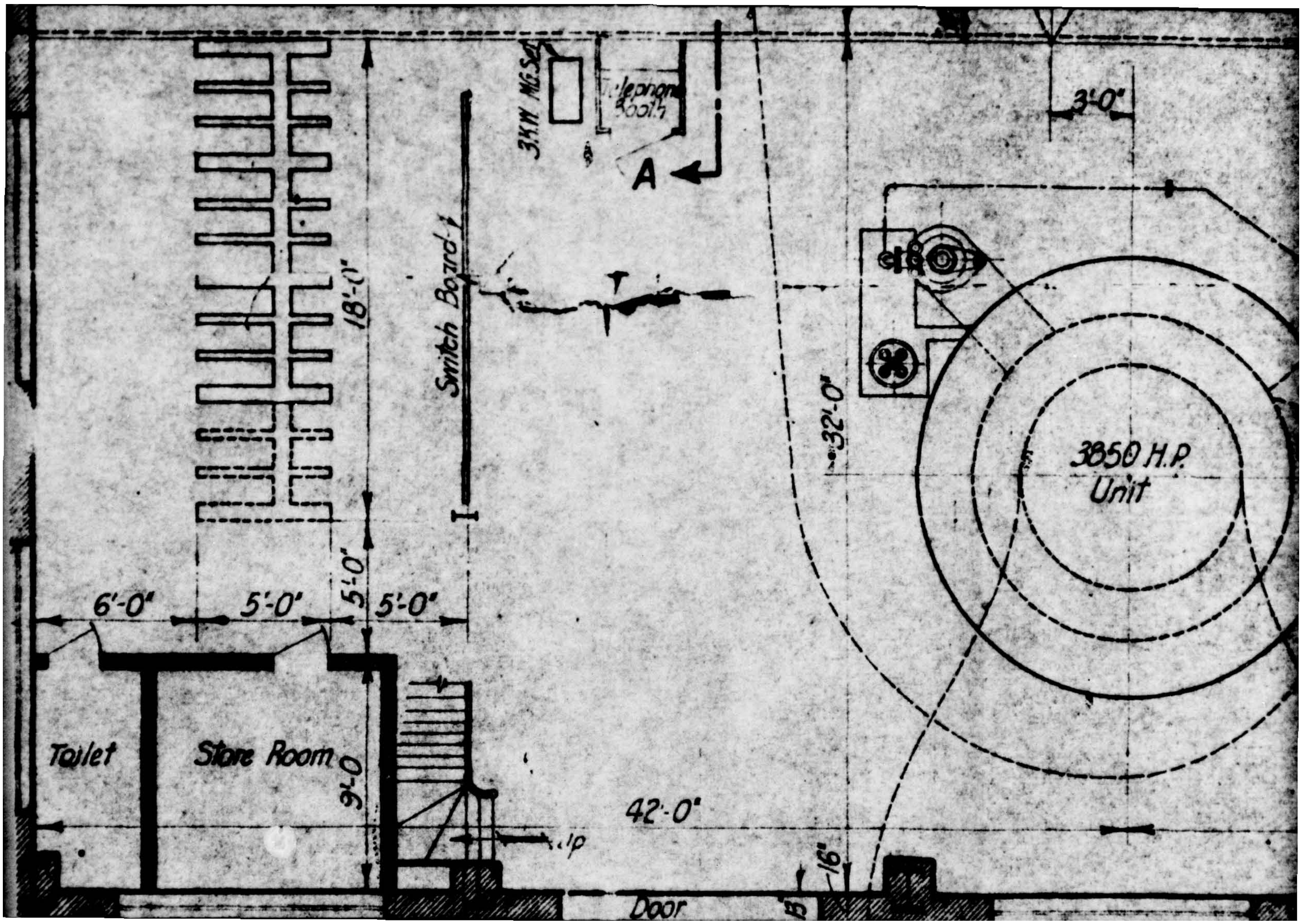


16'-0"
9'-6"
Transformer
Oil Switch
10'-6"
Elev. 142.0'
Elev. 140
CROSS SECTION "B-B"
4'-0"
10'-0"
3'-0"
10'-0"
4'-0"

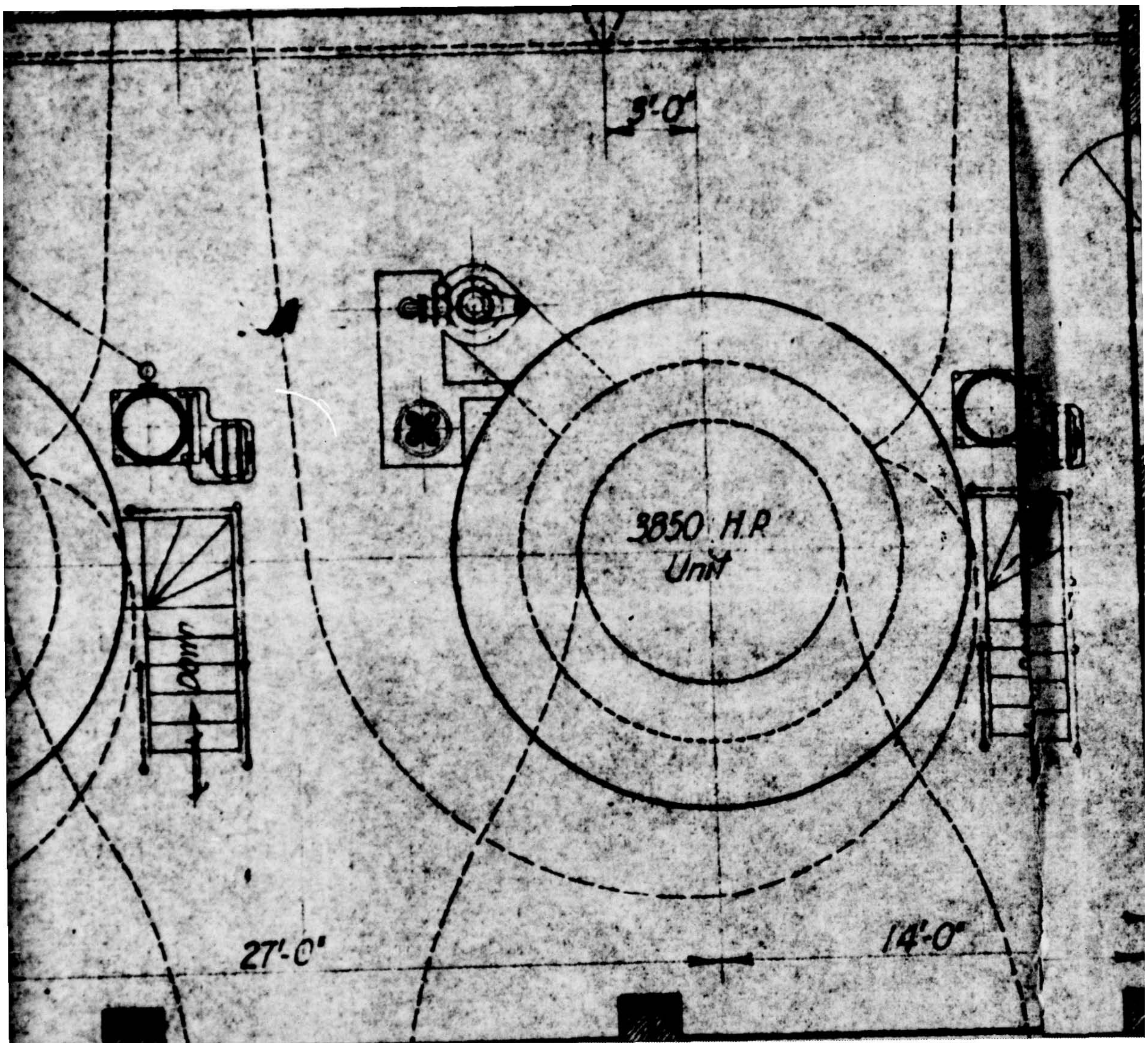


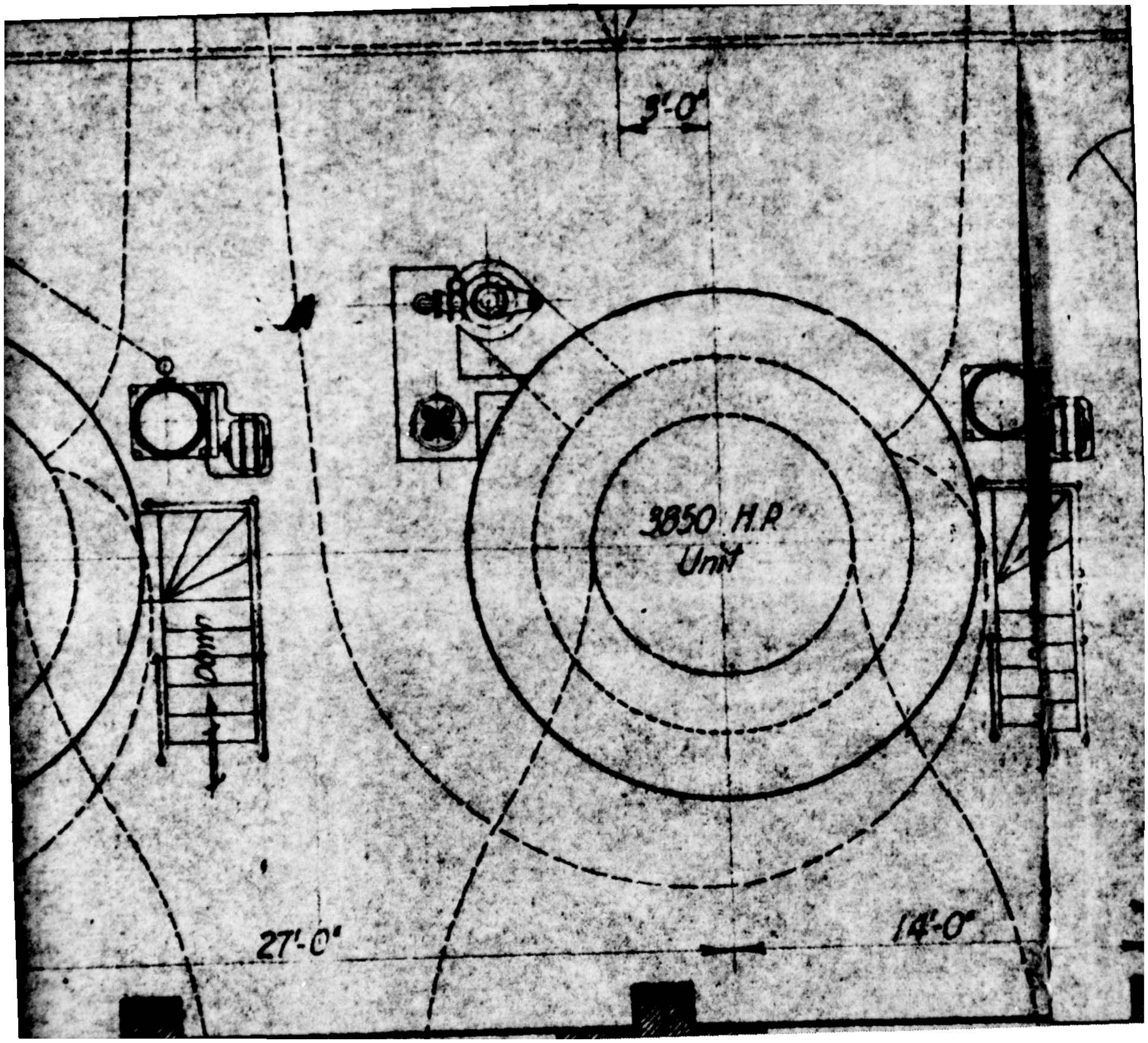
3
Elev. 172.04
Flashboards
Flashboard Socket
18'0" Radius
Bucket if required
142.0
7'-0"

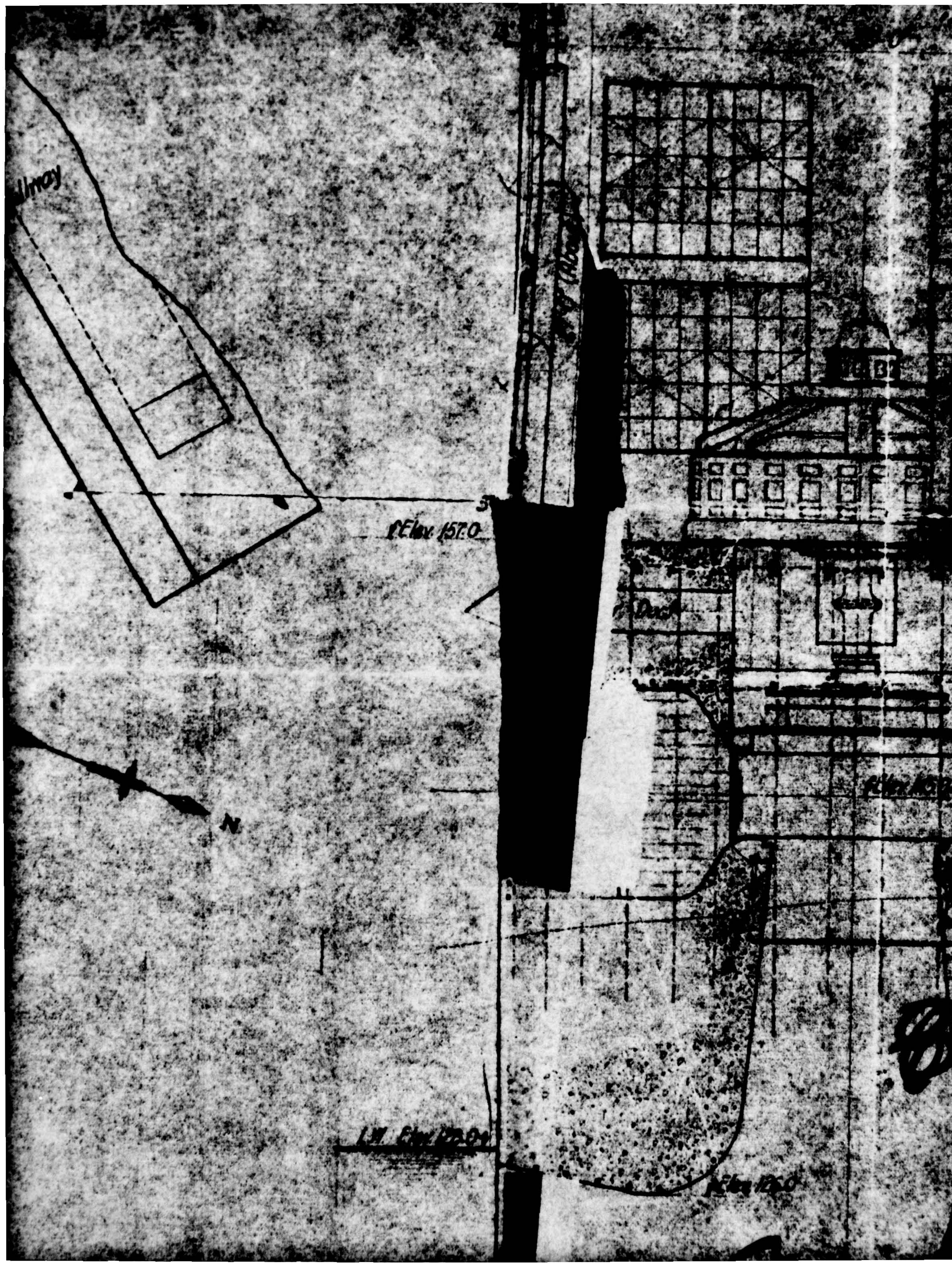

4



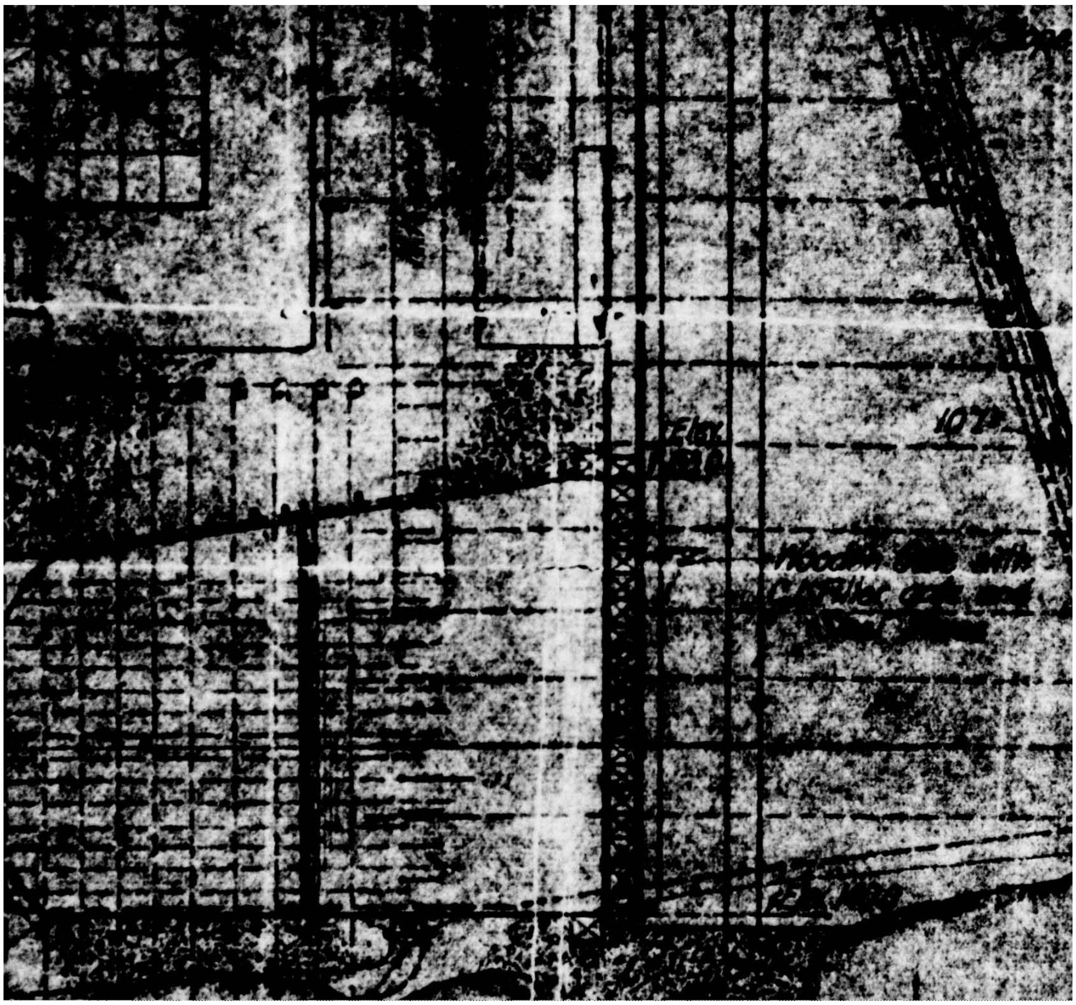
3.K.W. M.G. Set
Telephone Booth
A
Switch Board
18'-0"
32'-0"
3'-0"
3850 H.P. Unit
6'-0"
5'-0"
5'-0"
5'-0"
Toilet
Store Room
9'-0
42'-0"
16"
Door

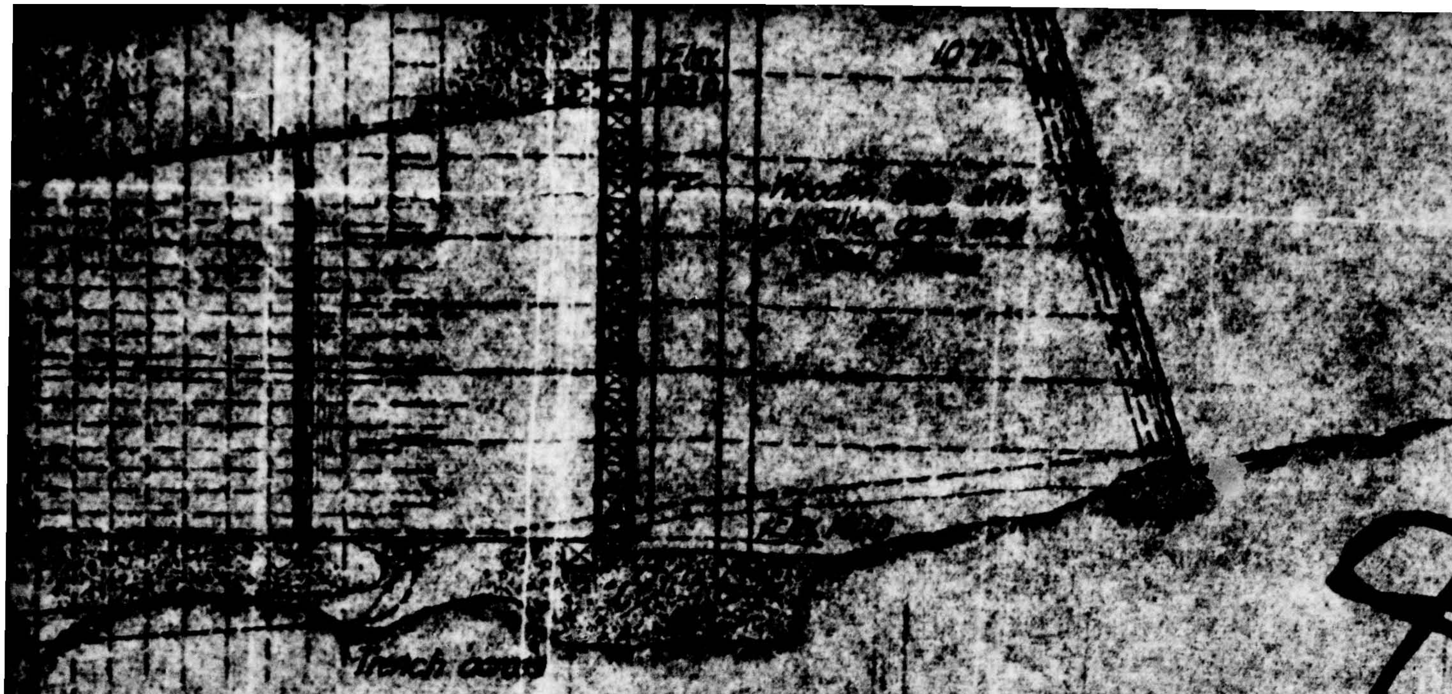

3'-0"
3850 H.P.
Unit
DOWN
27'-0"
14'-0"


3'-0"
3850 H.P.
Unit
DOWN
27'-0"
14'-0"



Elev. 157.0
Deck
N


Elev.
Wooden
with



8

SECTION OF POWER HOUSE

UNITED HUDSON ELECTRIC CORPORATION

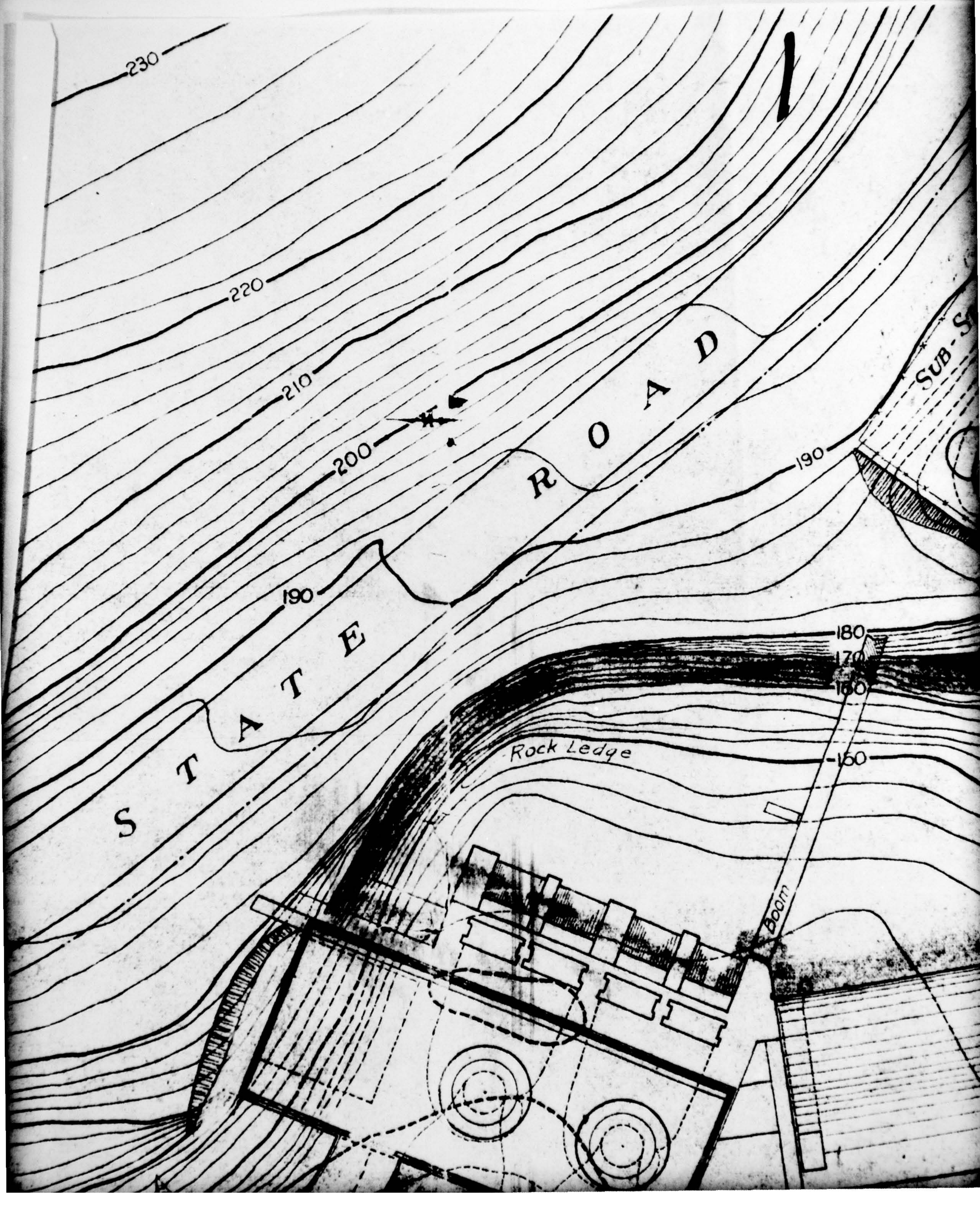
GLENVILLE DEVELOPMENT

PLAN AND SECTIONS OF POWER HOUSE AND DAM

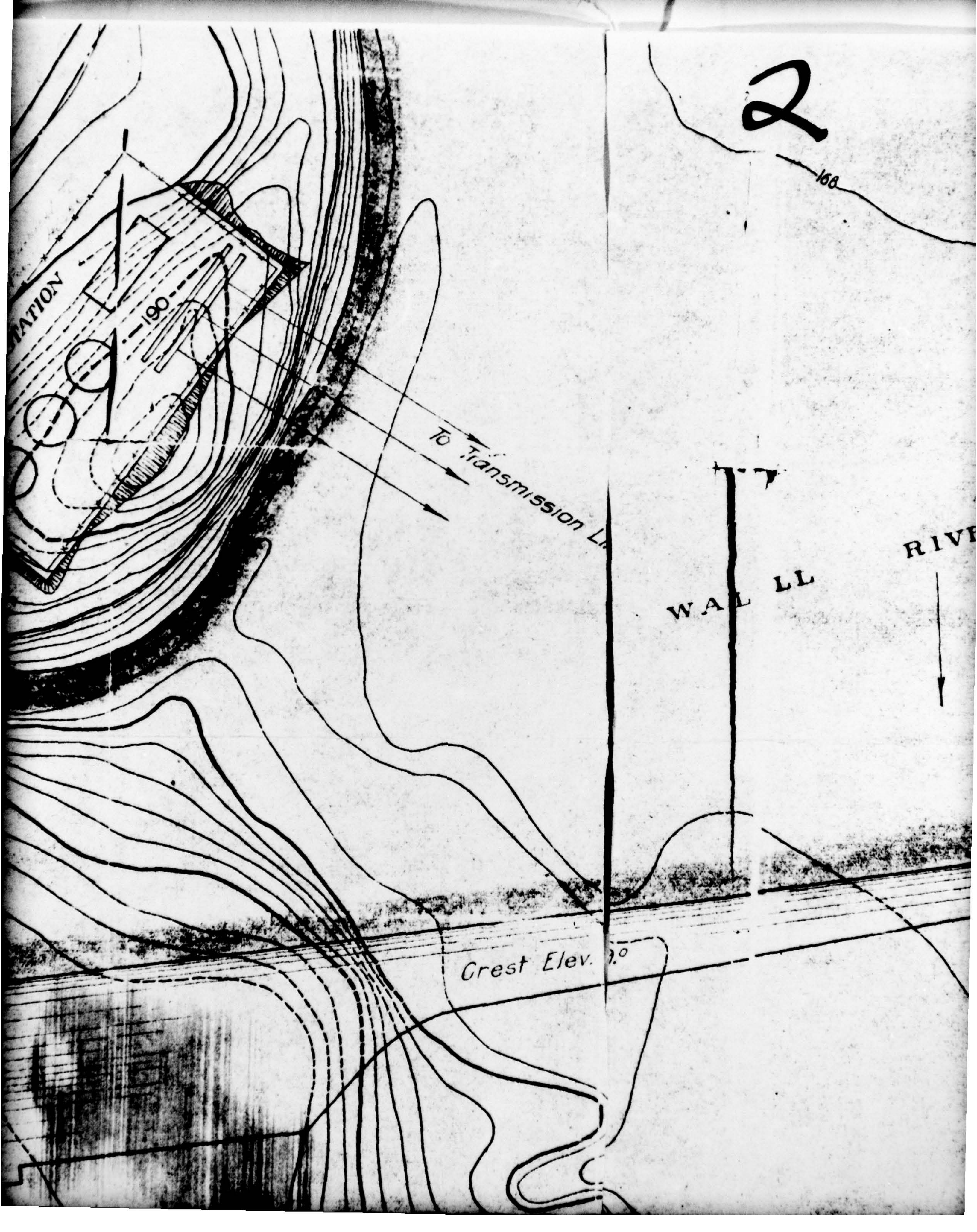
THE J.G. WHITE ENGINEERING CORPORATION

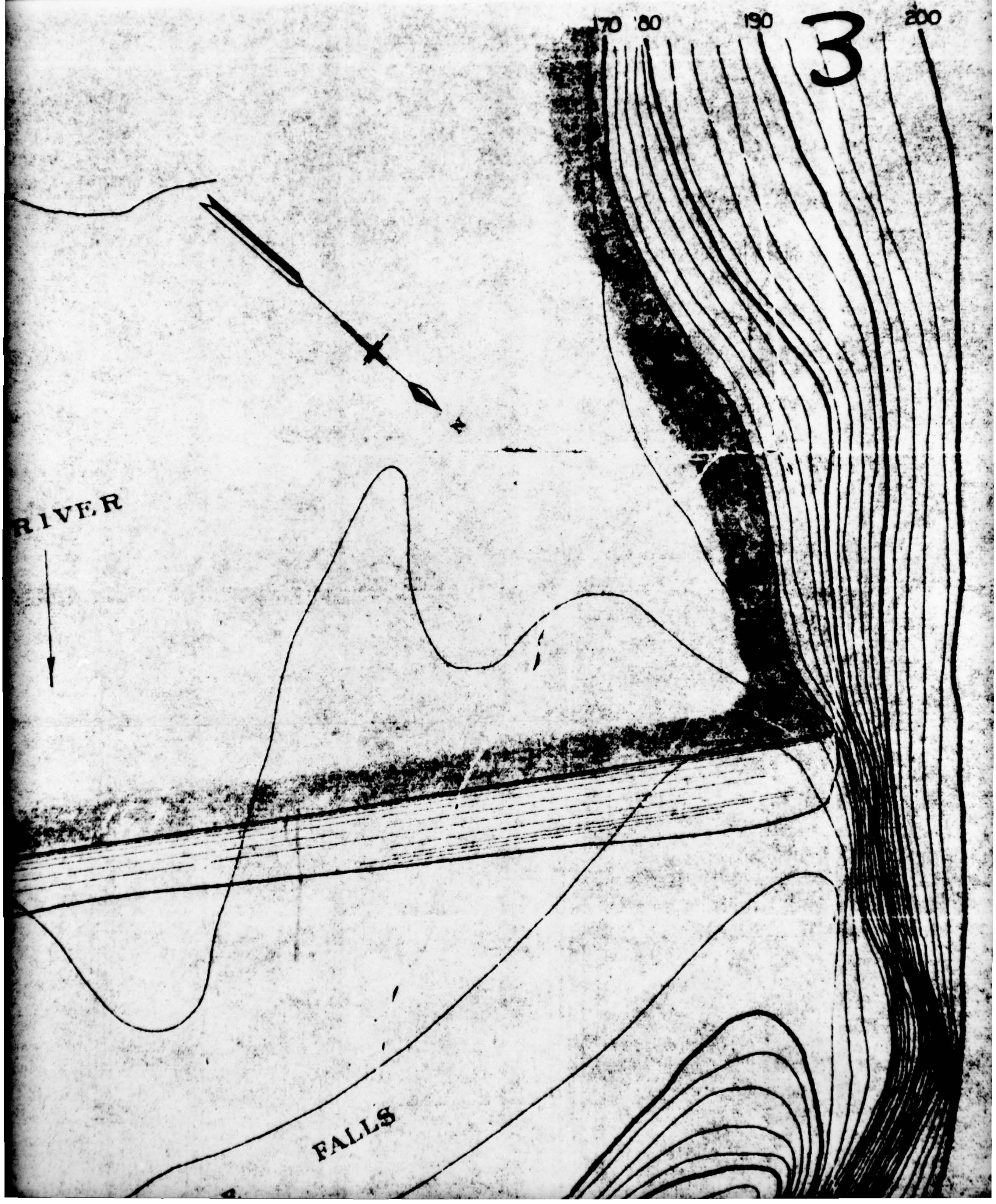
ENGINEERS AND CONTRACTORS

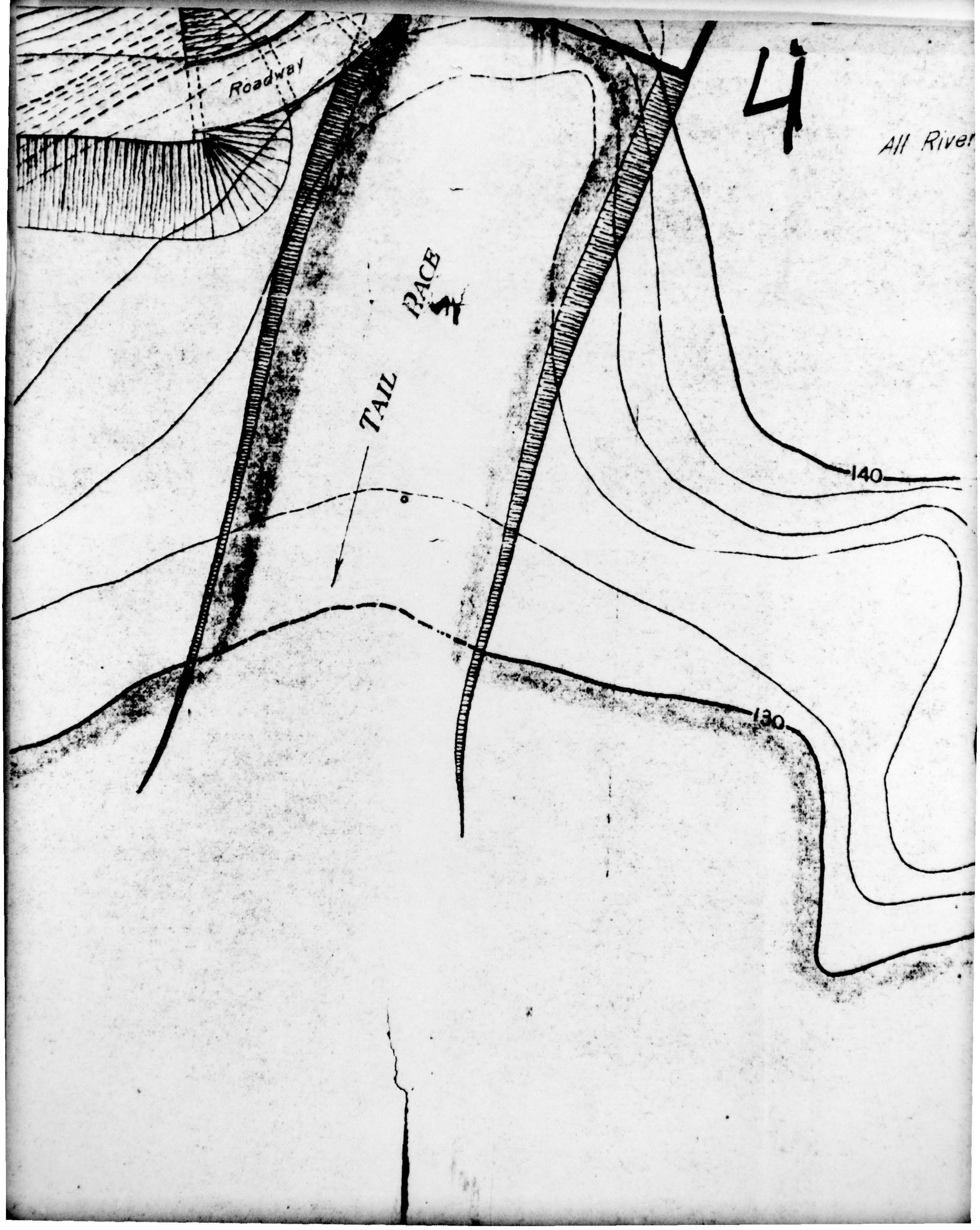
APPROVED ______ ENGINEER DATE August 7th 1919

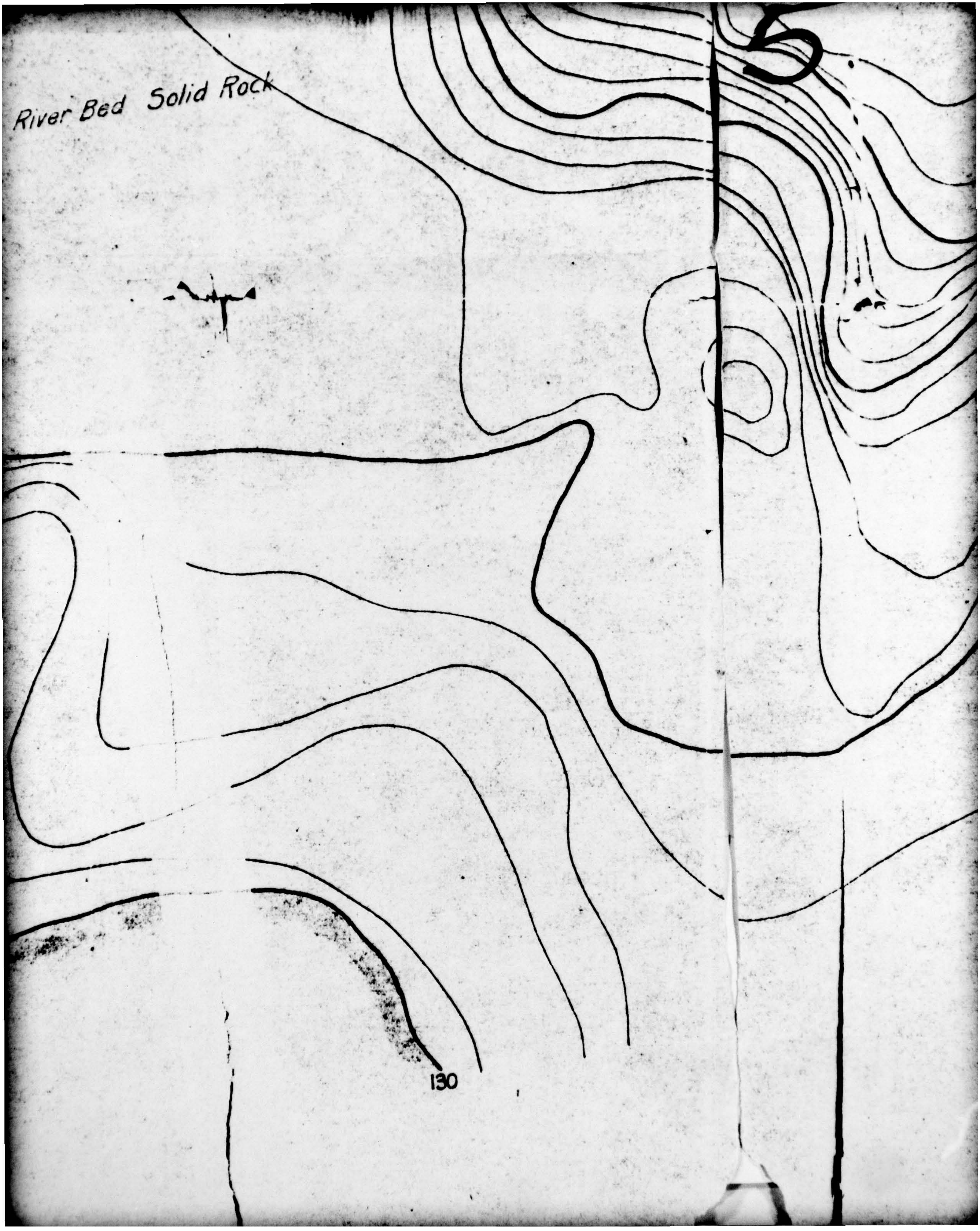


230
220
210
200
190
190
STATE ROAD
SUB-S
180
170
160
160
Rock Ledge
Boom

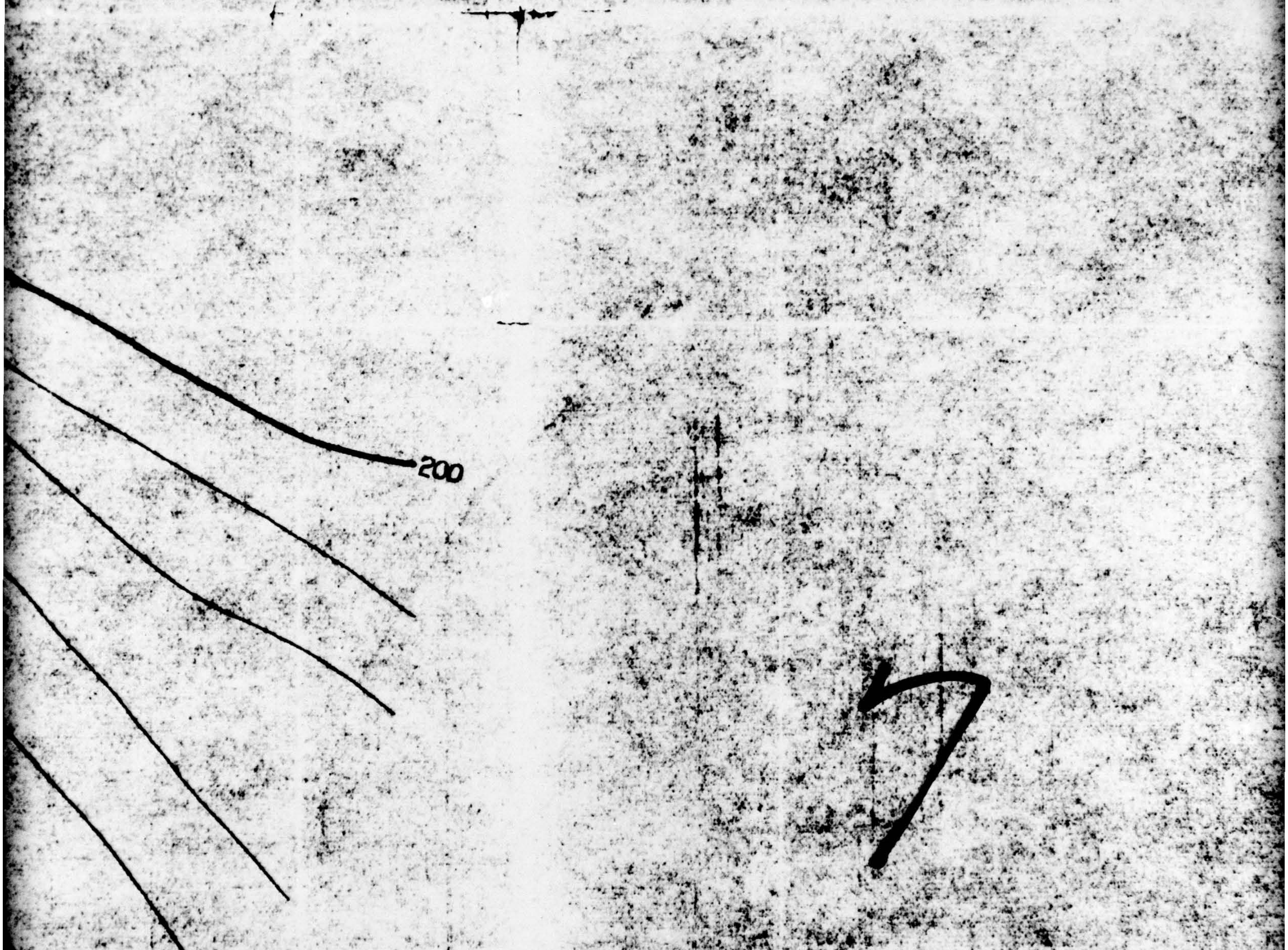

2
168
190
To Transmission Li
WAL
LL
Crest Elev.


170
180
190
3
200
RIVER
FALLS


4
Roadway
All River
TAIL RACE
140
130


River Bed Solid Rock
5
130


DASHVIL
160
150
150
170
180
190
200
140
150
160
170
180



UNITED HUDSON ELECTRIC CORPORATION

DASHVILLE DEVELOPMENT

CONTOUR MAP AND GENERAL LOCATION PLAN

THE J.G. WHITE ENGINEERING CORPORATION

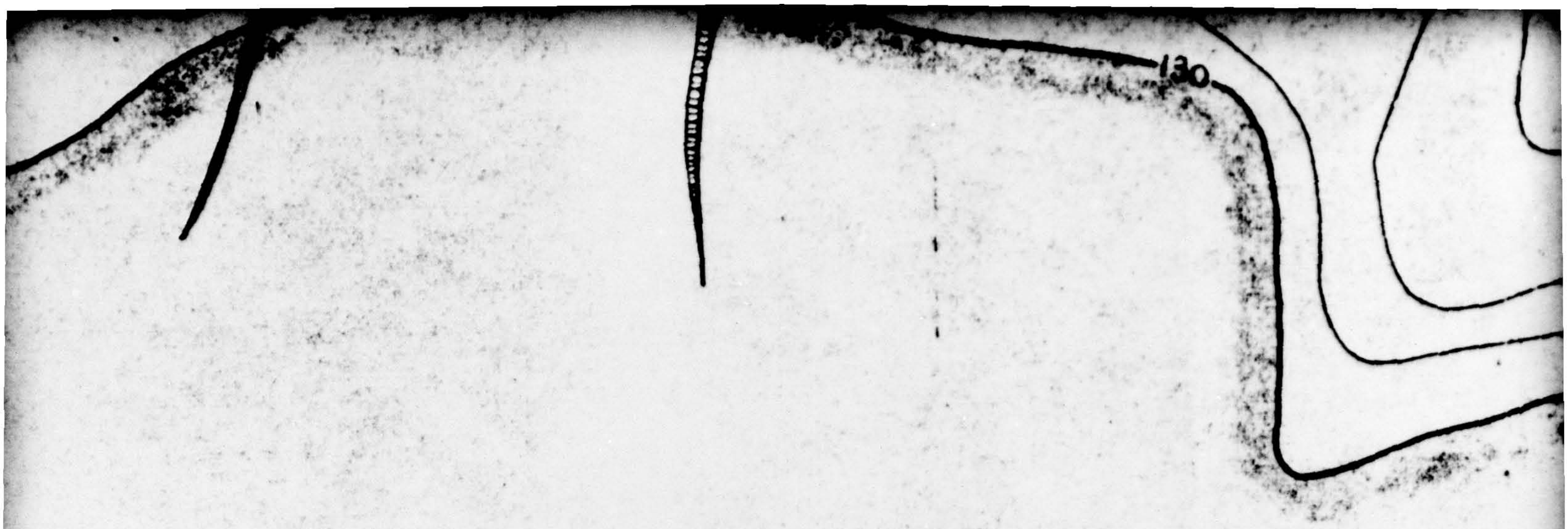
ENGINEERS AND CONTRACTORS

APPROVED [illegible] ENGINEER DATE MAY 29th/19

MAIN OFFICE – 43 EXCHANGE PLACE N.Y.

AUTHORIZATION NO 5300

SCALE 1" = 20'-0"



Note: Site located in Rosendale and Esopus Twps,
Ulster Co. N.Y., about 1 mile above Rifton.

8